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BUREAU OF PUBLIC ROADS

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A TAR SURFACE-TREATED ROAD IN NORTH CAROLINA

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G. P. St. CLAIR, *Editor*

Volume 14, No. 1 March, 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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TAR SURFACE TREATMENT OF LOW COST ROADS

Report of a Cooperative Study by the Bureau of Public Roads and Representatives of the Tar Industry

BECAUSE of the extensive application of surface treatment to the construction of low cost roads and the need for obtaining additional information on this subject, representatives of the tar industry and the Bureau of Public Roads have recently made a field and laboratory study, in North Carolina, of surface-treated roads on which tar was used. The particular type of treatment investigated consists essentially of the application of a light tar to the untreated road or prepared base, followed by an application of hot tar with a cover of mineral aggregate. A third application of tar, sometimes called the seal, is applied with a mineral aggregate cover after the surface has been subjected to traffic for a period of time. This method of treatment is designed to provide a wear-resisting surface which will adequately protect the base from deterioration and which can be economically maintained in a satisfactory condition.

The development of the double surface treatment as practiced in North Carolina resulted from early experiments in which a light tar was used on gravel or macadam bases. These tar-treated surfaces were very satisfactory for some time, as the tar penetrated into the road, allaying the dust and producing temporarily a bonded surface. However, under summer conditions, these surfaces disintegrated under traffic, and only by repeated re-treatments could the road surface be kept in good condition.

Later, more viscous tars applied hot were used. It was found that these hot tars produced a more durable surface on clean, hard bases but would not penetrate any existing dust film on the road. Because of lack of bond with the base, the road surfaces often corrugated badly or were picked up by traffic.

In order to obtain the advantages of both materials, the double surface treatment was developed, in which cold application tar was used as a prime and hot application tar for the second coat. In this way a wearing surface of greater thickness and stability was developed, which did not pick up under traffic but remained plastic and smooth under a wide range of weather conditions. Later, this same method of surface treatment was applied to topsoil and sand-clay surfaces.

A Tar-Treated Surface in Good Condition

This report embraces a detailed study of 19 projects in North Carolina, where approximately 1,200 miles of this form of treatment have been built on many different kinds of bases and under varying subgrade conditions. Inspection of a considerable mileage of this type of construction indicated that these projects were representative of North Carolina conditions. They were situated in the Atlantic coastal section, the Piedmont section, and the mountainous section in the western part of the State and were selected to cover a wide range of soil and climatic conditions as well as methods and materials of construction.

The detailed study of each project included an inspection to determine the appearance, condition, and riding

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qualities of the surface. Samples of bituminous mat, base, and subgrade representative of typical conditions were taken for analysis. For the purpose of this report the term "mat" is intended to describe the mixture of bituminous material and mineral aggregate immediately above the base. Information relative to details of construction, the type of maintenance employed and the amount of traffic carried was furnished by the State Highway Commission. Laboratory analyses of the bituminous mats were made and base material and subgrade samples were tested for grading and soil classification.

RESULTS OF SURVEY PROVE ADEQUACY OF TAR TREATMENT; IMPORTANCE OF SOIL STUDIES STRESSED

As a result of this survey the following conclusions seem warranted:

1. Surface treatment provides a satisfactory low-cost wearing surface on several types of base.

2. Surface treatment has the advantage over the thicker surface mats in that it is more pliable and can, therefore, adjust itself to the movement on a flexible base with a minimum amount of cracking.

3. New roads or base construction should be subjected to traffic for some time before being surface-treated, in order to produce adequate compaction and to determine the suitability of the base for surface treatment. Compaction under traffic is valuable as a means of detecting an excess of plasticity in the base, a condition which should be corrected.

4. In order to avoid failures in surface treatments resulting from unstable bases, research should be directed toward finding a means of reducing the plastic properties of the clay present, particularly in the natural soils.

5. A priming application is desirable as a final preparation of the base, not only to stabilize and waterproof the top but also to insure a bond between the base and the surface mat.

6. Tar of light consistency, because of its excellent penetrating and binding properties, makes a very satisfactory prime or stabilizing material.

7. Hot application tars are more satisfactory for the second application; medium and heavy cold application tars have many advantages in seal and re-treatment work.

8. A tough aggregate should be used as cover material in order to reduce crushing under traffic to a minimum so that the original texture of the surface and the stability of the mat may be retained.

9. Very fine sand or roadside material should not be used for blotting purposes because it has a tendency to produce either a brittle or an unstable surface.

10. A seal application is highly desirable as the final step in insuring a dense and waterproof surface.

11. Cold application tar is preferable to hot application tar in re-treatment work because it permits dragging and blading of the cover stone to produce a smoother surface. The finished road shows less tendency to bleed and is likely to retain its original texture for a longer time.

12. The principal requirement of bases for surface treatment is that they shall neither soften nor undergo appreciable change in volume with current changes of climatic or ground moisture conditions.

13. Soil tests serve to disclose those characteristics of the base material which materially affect the performance of surface treatments. In this connection the mechanical analysis is of assistance but by itself is not adequate for distinguishing good from poor base materials.

14. Determining the suitability of base soils according to their physical characteristics instead of by rule of thumb methods may show that the natural road soil, either alone or in some combination with other local soils, will serve as well or better than the more expensive base materials of recognized value.

15. A content of clay binder as low as 5 per cent has proved satisfactory in surface-treated bases. However, clays with capillary and cohesive properties as high as those of average or "statistical" soils[1] and shrinkage properties considerably lower, may not prove detrimental in base mixtures even when present in amounts as great as 20 per cent. More than 10 per cent of highly plastic clay and all cases of highly micaceous material should be regarded with suspicion.

16. Some method of correcting unsatisfactory base material may often prove desirable. Fine sand is used satisfactorily as an admixture to reduce clay content. Coarse material has not much value for this purpose and may be used to better advantage as a top course. The best method and material to use in stabilizing a soil will depend upon the character of the particular soil or base material. Often a thin top course of suitable material will increase the stability of the base sufficiently. In other cases, it may be advantageous to use a tar primer or to mix tar into the base.

VARIOUS BASES USED FOR SURFACE TREATMENT

The essential requirements of bases for surface treatment are that they shall carry the load imposed upon them and shall remain stable irrespective of moisture conditions without undergoing any change sufficient to injure the surface treatment. Among the more important materials used in base construction are the local topsoils, sand clay, sand-clay-gravel, shell rock, and various crushed materials. In North Carolina, topsoil and sand-clay, because of their greater prevalence, have been used more extensively than other materials in this type of construction.

The bases which were used on the sections discussed in this report were, with one exception, originally constructed to serve as topsoil, sand-clay, or sand-clay-gravel roads. The exception was a base of sand-clay-gravel built to be surface-treated immediately.

The topsoil and sand-clay used for surfacing consisted of a mixture of sand and clay either obtained from local deposits or produced by incorporating sand or clay with the natural soil. The surfaces were usually built with a compacted thickness of 6 to 9 inches at the center and feather-edged to the full width of the roadway.

The gravel surfacing material was a mixture of gravel, sand, and clay binder obtained largely from pits or stream beds. It was hauled to the roadway, spread, bladed, and compacted under traffic. The cross section was either of uniform thickness for a definite width or tapered to the full width of the roadway. The compacted thickness of the course ranged from 4 to 10 inches, depending on the character of the subsoil.

In the mountainous areas where subsoil and moisture conditions were unusually bad a subbase, made up of hammer-broken field stone, approximately 6 inches thick, was built before the construction of the gravel course. Projects 13, 14, and 15 were of this type.

[1] The significance of the term "statistical soils" is discussed on p. 15. See also PUBLIC ROADS, vol. 12, No. 5, July, 1931, p. 131.

CONDITIONING THE ROAD FOR SURFACE TREATMENT

Before applying the surface treatment it was generally necessary to do considerable work in the preparation of the base. The particular method followed and amount of work required depended on its condition.

If the base course was of sufficient thickness and composed of satisfactory materials the preparation generally consisted of light blading or dragging to obtain a smooth, uniform surface. If the thickness was insufficient new material was added and worked into the base course to produce a uniform andcompacted condition of the surface.

A layer of crushed stone screenings was applied to the surface of the south portion of project 19, prior to surface treatment, to reduce the plastic condition of the topsoil road.

On projects 4 and 5, where a loose and nonuniform condition of the topsoil and sand-clay existed, the soils were stabilized previous to treatment by road-mixing the soil with tar to form a compacted mat 2 to 3 inches in thickness.

With the exception of project 15, the bases were subjected to traffic for a considerable time prior to surface treatment. The compaction and bond obtained depended upon their time in service and the character of the materials used in their construction.

LOOSE AGGREGATE ON SURFACE OF BASE DISTRIBUTED AND ROLLED IN PREPARATION FOR APPLICATION OF PRIMER

LOOSE AGGREGATE BLADED UNIFORMLY AND READY FOR APPLICATION OF PRIMER

FIGURE 1.—SAND-GRAVEL BASES READY FOR SURFACE TREATMENT

Where the material contained excess clay, fine sand was often added to reduce the clay content. Coarse material, such as gravel or crushed stone, was also used in some cases to improve the condition of the base. When so used it was generally applied as a thin surface course rather than being incorporated into the base material. If the surface was loose because of insufficient or unsatisfactory binder material, as was the case when comparatively clean sand-gravel was used in the base course, a somewhat heavier application of tar primer was used.

On project 12, where the untreated surface cracked excessively in dry weather and was slippery in wet weather, a ¾-inch layer of creek sand was mixed into the top 4 inches of the road surface to reduce the plastic properties of the topsoil and to provide a more stable base for the subsequent surface treatment.

On the sand-clay portion of project 18 a layer of crushed stone 2½ inches to 1 inch in size, was rolled into the surface to stabilize its loose, unbonded condition.

Immediately prior to surface treatment the bases were generally dragged or bladed lightly to obtain a smooth uniform surface. On those projects where the surface was well bonded it was swept. If the surface was loose, as on the projects where relatively clean sand-gravel was used, the loose material was spread uniformly over the surface before priming.

Table 1 gives the mechanical analysis of sand-clay-gravel materials taken from the base courses of a number of projects, determined by the method commonly used in studying materials to be used for base construction. It is evident, however, that this method does not provide sufficient information, as it does not distinguish between silt and clay nor does it indicate the character of the clay.

Tables 2 and 3 give more definite and complete information on the properties of the various materials

TABLE 1.—*Mechanical analysis of material taken from the base courses of a number of typical sand-clay-gravel roads which have been surface-treated*

Laboratory number	6206	6063	6064	6060	6061	6062	6203	6196	6197	6199	6198	6201
Project number	1	2	2	7	7	7	11	13	14	15	16	16
Hole number	35	10	12	7	8	9	23	18	19	21	20	22
Total retained on—	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*
3-inch screen	------	------	------	------	------	------	10	9	0	7	0	------
2½-inch screen	------	------	------	------	------	------	19	16	45	12	5	0
2-inch screen	------	------	------	0	0	0	23	23	51	19	22	14
1½-inch screen	------	------	------	4	3	8	28	26	57	36	29	28
1¼-inch screen	0	------	------	4	3	8	32	30	60	39	32	30
1-inch screen	2	------	0	7	8	12	37	33	63	44	41	32
¾-inch screen	4	0	3	13	11	12	42	39	68	51	46	39
½-inch screen	6	2	9	17	14	15	49	45	73	57	52	45
¼-inch screen	14	11	17	22	21	20	58	54	77	65	57	52
No. 10 sieve	27	23	27	29	29	27	66	65	80	73	62	58
No. 20 sieve	43	39	37	40	42	39	75	75	82	80	67	63
No. 30 sieve	49	48	41	45	49	41	79	79	83	83	69	65
No. 40 sieve	58	60	47	51	57	44	84	84	84	86	72	68
No. 50 sieve	66	69	52	56	64	48	89	89	86	88	74	72
No. 80 sieve	73	78	61	62	72	59	94	94	89	90	79	80
No. 100 sieve	75	79	67	63	------	61	95	95	90	91	81	82
No. 200 sieve	82	84	75	66	79	70	97	97	93	93	86	88
Percentage of original sample over ¼ inch	14	11	17	22	21	20	58	54	77	65	57	52
Percentage of original sample under ¼ inch	86	89	83	78	79	80	42	46	23	35	43	48
Percentage of silt and clay (by washing)	18.1	16.0	24.4	33.6	22.2	30.0	3.0	2.4	6.9	7.5	13.9	11.3
Condition of bituminous surface December, 1931	Good.	Good.	Cracked.	Good.	Good.	Wavy.	Good.	Wavy.[1]	Slightly wavy.[1]	Good.	Good.	Slightly cracked

[1] Base corrugations not removed before surface treating.

TABLE 2.—*Analyses of base and subgrade materials on projects having sand-clay-gravel base*

Identification, location, and description of sample				Mechanical analysis							Physical characteristics of material passing No. 40 sieve						Soil group	Condition of wearing surface, December, 1931
Project No.	Hole No.	Laboratory No.			Particles larger than 2 millimeters	Particles smaller than 2 millimeters, per cent					Liquid limit	Plasticity index	Shrinkage		Moisture equivalent			
		Base	Subgrade			Coarse sand [1]	Fine sand [2]	Silt [3]	Clay [4]	Colloids [5]			Limit	Ratio	Centrifuge	Field		
1	36	S–6369	------	4 inches thick	24	36	35	8	21	16	34	19	18	1.8	17	23	A–2 plastic	Badly cracked.
	35	S–6206	------	3½ inches thick	30	57	20	8	15	12	32	15	20	1.7	18	24	do	Good.
	35	------	S–6194	14 inches thick / 4 inches below surface	0	7	41	37	15	8	17	3	15	1.9	17	16	do	
2	10	S–6063	------	3 inches thick	30	58	20	8	14	12	29	13	19	1.7	19	24	do	Do.
	10	------	S–6057	4½ inches thick / 4½ inches below surface	1	15	55	15	15	11	19	0	18	1.8	8	18	A–3	
	12	S–6064	------	5 inches thick	26	32	35	11	22	17	34	18	17	1.8	18	22	A–2 plastic	Cracked.
	12	------	S–6059	2 inches thick / 6 inches below surface	0	47	37	7	9	6	19	0	21	1.7	7	20	A–3	
7	7	S–6060	------	5 inches thick	30	34	19	11	36	30	48	28	18	1.7	30	27	A–7	Good.
	7	------	S–6054	6¼ inches below surface	1	11	20	21	48	32	61	29	31	1.5	34	42	A–5	
	8	S–6061	------	4¾ inches thick	34	52	20	15	13	5	12	2	29	1.5	12	11	A–2	Do.
	8	------	S–6055	6 inches below surface	0	6	23	28	43	27	46	24	16	1.8	34	32	A–7	
	9	S–6062	------	5¾ inches thick	32	30	36	25	9	5	18	4	18	1.8	11	17	A–2	Wavy.
	9	------	S–6056	3 inches thick / 7 inches below surface	2	26	29	22	23	18	29	14	16	1.8	18	21	A–2 plastic	
11	23	S–6203	------	4½ inches thick	71	66	25	5	4	3	21	0	28	1.5	10	25	A–3 highly micaceous	Good.
	23	------	S–6173	6½ inches below surface	1	17	37	16	30	18	33	15	21	1.7	28	25	A–2 plastic	
13	18	S–6196	------	6 inches thick	66	68	24	3	5	3	23	0	29	1.5	7	28	A–3	Wavy; base corrugations not removed before treatment.
	18	------	------	Broken stone 6 inches thick														
	18	------	S–6170	13 inches below surface	0	20	47	15	18	14	28	6	24	1.6	27	31	A–2 plastic; highly micaceous.	
14	19	S–6197	------	4 inches thick	45	25	47	17	11	6	25	7	24	1.6	14	24	do	Slightly wavy; base corrugations not removed before treatment.
	19	------	------	Broken stone 6 inches thick														
	19	------	S–6171	Under 6-inch stone base	32	9	29	32	30	9	44	11	33	1.4	37	40	A–5 highly micaceous	
15	21	S–6199	------	4 inches thick	38	46	29	18	7	4	26	0	(6)	-----	22	28	A–2	Good.
	21	------	------	Broken stone 6 inches thick														
	21	------	S–6200	Under 6-inch stone base	55	21	32	29	18	8	26	7	23	1.7	23	25	A–2 plastic; appreciable mica.	
16	20	S–6198	------	7½ inches thick	53	31	42	16	11	5	19	0	23	1.7	13	19	A–2 slightly micaceous	Good.
	20	------	S–6172	8½ inches below surface	0	17	31	23	29	17	43	19	26	1.6	37	33	A–7	
	22	S–6201	------	12 inches thick	58	41	38	13	8	4	22	0	24	1.6	10	21	A–3	Slightly cracked.
	22	------	S–6202	13 inches below surface	44	29	43	19	9	7	25	0	29	1.5	17	27	A–2 highly micaceous	

[1] 2.0 to 0.25 millimeter.
[2] 0.25 to 0.05 millimeter.
[3] 0.05 to 0.005 millimeter.
[4] Smaller than 0.005 millimeter.
[5] Smaller than 0.001 millimeter.
[6] No shrinkage. Soil swelled.

TABLE 3.—*Analyses of base and subgrade materials on projects having topsoil or sand-clay base*

TOPSOIL BASE PROJECTS

Project No.	Hole No.	Laboratory No. Base	Laboratory No. Subgrade	Identification, location, and description of sample	Mechanical analysis: Particles larger than 2 millimeters	Particles smaller than 2 millimeters, per cent: Coarse sand [1]	Fine sand [2]	Silt [3]	Clay [4]	Colloids [5]	Physical characteristics of material passing No. 40 sieve: Liquid limit	Plasticity index	Shrinkage: Limit	Shrinkage: Ratio	Moisture equivalent: Centrifuge	Moisture equivalent: Field	Soil group	Condition of wearing surface, December, 1931
	1	S-6043		7 inches thick	0	40	40	11	9	4	16	0	20	1.7	5	18	A-3	Lean.
	1		S-6044	6 inches thick, 10¼ inches below surface.	4	32	24	21	23	16	28	13	17	1.8	21	20	A-2 plastic	
4	2	S-6045		1 inch thick	3	46	26	16	12	5	16	3	16	1.8	14	15	A-2	Spongy and cracked.
	2		S-6046	5½ inches below surface	3	23	20	23	34	21	49	22	27	1.6	40	36	A-7	
	3	S-6047		6½ inches thick	0	37	40	14	9	4	14	0	19	1.8	7	15	A-3	Good.
	3		S-6048	2½ inches thick overlying sand soil.	3	40	30	14	16	11	21	8	18	1.8	15	18	A-2 plastic	
	4	S-6049		18 inches thick	1	52	22	12	14	8	15	5	13	1.9	11	13	A-2	Do.
	5	S-6050		2 inches thick	4	51	18	14	17	10	20	8	17	1.8	18	16	A-2 plastic	Do.
6	5		S-6051	8 inches below surface	4	37	22	20	21	16	42	16	29	1.5	35	34	A-5	
	6	S-6052		8½ inches thick	2	41	32	9	18	13	13	2	14	1.9	8	12	A-3	Do.
	6		S-6053	10 inches below surface	1	21	26	19	34	26	47	20	28	1.5	35	36	A-7	
	14	S-6165		6 inches thick	0	20	35	29	16	6	19	4	17	1.8	18	19	A-2 plastic	Rough and wavy.
8	33	S-6190		6½ inches thick	0	33	21	29	17	11	20	5	19	1.9	18	19	A-2 highly micaceous	Badly cracked.
	33		S-6205	5½ inches thick	59	21	30	27	22	14	27	11	25	1.7	23	21	A-2 plastic	
	33		S-6191	13 inches below surface	0	7	26	39	28	18	47	23	23	1.6	38	34	A-7	
	15	S-6166		7 inches thick	13	34	30	22	14	8	14	3	15	1.9	14	14	A-2 plastic	Good.
9	15		S-6167	8¼ inches below surface	0	18	12	26	44	34	52	27	19	1.7	37	39	A-7	
	16	S-6168		7 inches thick	0	47	22	17	14	10	18	4	17	1.8	13	16	A-2 plastic	Slightly cracked.
	16		S-6169	7¾ inches below surface	0	25	23	23	29	18	41	21	19	1.7	29	28	A-7	
	24	S-6174		8 inches thick	0	27	38	16	19	14	24	8	19	1.8	20	21	A-2 plastic	Good.
	24		S-6175	9 inches below surface	1	14	33	33	20	12	46	13	40	1.3	38	45	A-5 highly micaceous	
10	25	S-6176		5½ inches thick	23	18	27	32	23	13	34	12	25	1.6	27	29	A-4 appreciable mica and gravel.	Cracked.
	25		S-6204	6 inches below surface	0	14	51	17	18	14	42	12	36	1.4	25	41	A-5 highly micaceous	
	30	S-6186		7¼ inches thick	0	40	34	10	16	9	19	4	15	1.9	17	19	A-2 plastic	Good.
12	30		S-6188	8 inches below surface	2	25	26	19	30	24	46	19	29	1.5	28	36	A-5 highly micaceous	
	31	S-6187		5¼ inches thick	2	35	20	21	24	14	39	16	23	1.7	28	30	A-2 plastic	Slightly cracked.
	31		S-6189	6 inches below surface	0	32	33	23	12	8	53	0	47	1.2	34	55	A-5 highly micaceous	
17	29	S-6184		10 inches thick	0	39	33	13	15	12	18	6	16	1.9	17	16	A-2 plastic	Good.
	29		S-6185	11 inches below surface	1	17	24	18	41	33	53	24	25	1.6	33	36	A-7	
18	27	S-6180		4 inches thick	0	25	30	19	26	17	29	10	21	1.7	28	24	A-2 plastic	Rutted.
	27		S-6181	4¾ inches below surface	0	22	41	20	17	13	40	12	36	1.4	24	39	A-5 highly micaceous	
19	28	S-6182		6 inches thick	0	36	26	13	25	13	27	10	17	1.8	23	20	A-2 plastic	Good.
	28		S-6183	7 inches below surface	0	1	38	30	31	22	56	22	32	1.5	40	46	A-5 highly micaceous	

SAND-CLAY BASE PROJECTS

2	11	S-6058		2½ inches thick	1	10	65	10	15	11	18	0	21	1.7	8	18	A-3	Good.
3	13	S-6195		6¼ inches thick	0	60	20	7	13	10	23	10	16	1.9	13	18	A-2 plastic	Pitted.
	13		S-6164	7 inches thick 8 inches below surface.	0	25	33	32	10	4	16	0	20	1.7	12	12	A-2	
5	34	S-6192		10½ inches thick	0	55	12	7	26	23	39	18	23	1.6	24	28	A-2 plastic	Good.
	34		S-6193	12½ inches below surface	0	68	16	6	10	7	15	0	18	1.8	5	17	A-3	
	26	S-6177		4 inches thick. Stone veneer added.	0	56	20	10	14	10	23	7	19	1.7	20	21	A-2 plastic, some mica	Do.
18	26		S-6178	3½ inches thick 6½ inches below surface.	0	36	23	13	28	20	46	22	26	1.6	31	33	A-7 appreciable mica	
	26		S-6179	10 inches below surface	0	31	45	10	14	10	34	0	38	1.3	19	36	A-2 some mica	

[1] 2.0 to 0.25 millimeter. [2] 0.25 to 0.05 millimeter. [3] 0.05 to 0.005 millimeter. [4] Smaller than 0.005 millimeter. [5] Smaller than 0.001 millimeter.

used as bases on these projects, and similar information regarding the subgrade soils. Table 2 gives the mechanical analyses and also the results of the physical tests performed on samples taken from the sand-clay-gravel projects. Table 3 gives corresponding data on samples taken from the topsoil and sand-clay projects. The significance of these test results in determining the suitability of the materials is discussed subsequently in this report.

CONSTRUCTION OF THE SURFACE TREATMENT

A prime coat is the initial application of a tar having low viscosity to the original untreated road surface for the purpose of hardening and moistureproofing the upper portion of the existing road. It produces a dust-free surface to which the subsequent application of more viscous tars will readily adhere.

In cases where the base was well bonded the surface was swept to remove dust and other objectionable material, prior to the application of the primer. Where the surface of the base was loose sweeping was omitted and the loose material was spread uniformly and sometimes rolled. Figure 1 shows two examples of such a surface ready for priming. The two upper views of Figure 2 illustrate the excellent surface which can be obtained by proper prime treatment of a loose base. The material used as a primer was a cold application tar, generally having a specific viscosity [2] of 8 to 13 at 40° C. However, 13-to-18 and 18-to-25 viscosity tars were used where the surfaces were open or loose. The amount of primer used on most projects was a third of a gallon; on others as much as 0.4 or 0.5 gallon was used. On projects 4 and 5, where the tar priming material was used to construct a mixed base; 1.2 and 1.5 gallons were used. The kinds and amounts of tar primer used on the various projects are given in Table 4, and typical specifications and test values are given in Table 5.

[2] Unless otherwise specified, statements regarding viscosity refer to the specific viscosity (Engler) at 40° C.

TABLE 4.—*Details of construction and re-treatments of projects studied*

CONSTRUCTION OF BASE

Project No.	1	2	3	4	5	6	7	8	9	10
Length in miles	8.33	9.432	10.3	5.0	6.7	8.5	15.841	5.0	7.955	9.4
Date constructed	December, 1927	December, 1927	April, 1928	July, 1923	1926	July, 1923	May, 1929	May, 1927	July, 1928	December, 1928.
Material	Sand-clay-gravel	Sand-clay-gravel	Sand-clay	Topsoil	Sand-clay	Topsoil	Sand-clay-gravel	Topsoil	Topsoil	Topsoil.
Width in feet	24	24	24	18	30	30	18	18	30	24.
Depth in inches (center)	4	4	4	10	10	10	6	6	9	6.
Subbase										
Surface behavior (Dry weather	Compacted	Compacted	Compacted	Raveled	Raveled	Raveled	Compacted	Compacted	Compacted	Compacted.
Surface behavior (Wet weather	Shallow ruts	Shallow ruts	Shallow ruts	Compacted	Compacted	Compacted	Shallow ruts	Shallow ruts	Shallow ruts	Slippery.

PREPARATION OF BASE

Preliminary preparation										
Condition at time of priming	Bonded, smooth	Bonded, smooth	Bonded, smooth.	Generally loose and nonuniform.	Generally loose and nonuniform.	Bonded, smooth.	Bonded, smooth	Bonded, smooth.	Bonded, smooth.	Bonded, smooth.

SURFACE TREATMENT [1]

Date constructed	October, 1928	July, 1928	April, 1929	July, 1928	1927	1928	September, 1929	July, 1928	September, 1928	May, 1923.
Prime coat:										
Grade of tar	TC-1	TC-1	TC-1	TC-3 [2]	TC-4 [2]	TC-1	TC-1	TC-1	TC-1	TC-1.
Gallons per square yard	⅓	⅓	⅓	1.2	1.5	⅓	⅓	⅓	⅓	⅓.
Second application:										
Grade of tar	TH-1	TH-1	TH-1	[3]	[3]	TH-1	TH-1	TH-1	TH-1	TH-1.
Gallons per square yard	⅓	⅓	⅓			⅓	⅓	⅓	⅓	⅓.
Cover material:										
Type	Granite	Granite	Limestone			Granite	Granite	Granite	Granite	Limestone.
Size	1 to ⅜ inch	1 to ⅜ inch	1¼ to ⅜ inch			1 to ⅜ inch	1 to ⅜ inch	¾ to ¼ inch	1 to ⅜ inch	1 to ⅜ inch.
Pounds per square yard	35	35	38			35		45		
Cost of double application	$2,191.78	$2,100.32	$1,574.02				$1,745.08	[4] $4,224	$1,946.97	$2,020.14.
Seal coat:										
Date of application	March, 1930	July, 1929	November, 1929	August, 1928	1928	1929	September, 1930	April, 1929	June, 1929	October, 1929.
Grade of tar	TC-4	TC-4	TC-5	TC-3	TC-4	TC-4	TC-5	TH-1	TC-4	TC-4.
Gallons per square yard	0.3	⅓	⅓	0.4	½	⅓	0.3	¼	⅓	0.3.
Cover material:										
Type	Granite	Granite	Granite	Granite	Granite	Granite	Granite	Granite	Granite	Limestone.
Size	¾ to ⅜ inch	¾ to ⅜ inch	¾ to 3/16 inch	1 to ⅜ inch	1¼ to ⅜ inch	¾ to ⅜ inch	¾ to ⅜ inch	¾ to ¼ inch	1 to ⅜ inch	¾ to ⅜ inch.
Pounds per square yard	20	20	20	20	35		20	25		16.

RE-TREATMENT

First re-treatment:										
Date								May, 1930		
Grade of tar								TH-1		
Gallons per square yard								0.3		
Cover material—										
Type								Limestone		
Size								¾ to ⅜ inch		
Pounds per square yard								32		
Second re-treatment:										
Date										
Grade of tar										
Gallons per square yard										
Cover material—										
Type										
Size										
Pounds per square yard										

CONSTRUCTION OF BASE

Project No.	11	12	13	14	15	16	17	18		19	
Length in miles	4.1	11.269	3.5	6.856	10.5	11.0	7.4	2.0	4.01	2.8	8.6
Date constructed	May, 1926	June, 1928	June, 1929	October, 1927	July, 1930	May, 1924	June, 1928	August, 1927		September, 1927	
Material	Sand-clay-gravel	Topsoil	Sand-clay-gravel	Sand-clay-gravel	Sand-clay-gravel	Sand-clay-gravel	Topsoil	Topsoil	Sand-clay.	Sand-clay.	Topsoil.
Width in feet	22	30	22	22	22	22	30	24	24	26	26.
Depth in inches (center)	10	9	5	5	5	4 to 12	9	9	9	9	9.
Subbase			6 inches field stone	6 inches field stone	6 inches field stone						
Surface behavior: Dry weather	Raveled	Raveled	Raveled	Raveled	Raveled	Raveled	Raveled	Compacted	Raveled		Compacted.
Surface behavior: Wet weather	Compacted	Compacted	Compacted	Compacted	Compacted	Compacted	Compacted	Rutted	Compacted		Slippery.

PREPARATION OF BASE

Preliminary preparation		Layer of sand worked into surface December, 1928.							Surface course 2 inches of crushed stone.		Screenings worked into surface.
Condition at time of priming	Loose	Bonded, smooth.	Loose, corrugated.	Bonded, corrugated.	Loose	Bonded, corrugated.	Bonded, smooth.	Bonded, smooth.			Bonded, smooth.

SURFACE TREATMENT [1]

Date constructed	October, 1926	June, 1929	October, 1930	August, 1928	May, 1930	August, 1927	August, 1928	October, 1928			October, 1928.
Prime coat:											
Grade of tar	TC-1	TC-1	TC-1	TC-1	TC-2	TC-1	TC-1	TC-1	TC-1		TC-1.
Gallons per square yard	0.4	⅓	⅓	⅓	0.4 to 0.5	⅓	⅓	⅓	0.4		⅓.
Second application:											
Grade of tar	TH-1	TH-1	TC-6	TH-1	TH-1	TH-1	TH-1	TH-1	TH-1		TH-1.
Gallons per square yard	0.4	⅓	⅓	⅓	0.4 to 0.5	⅓	⅓	⅓	0.4		⅓.
Cover material:											
Type	Granite	Limestone	Local stone	Limestone	Limestone	Limestone	Limestone	Limestone.			
Size	1 to ⅜ inch	1 to ⅜ inch	1 to ⅜ inch	1 to ⅜ inch	1 to ⅜ inch	⅝ to ⅜ inch	1 to ⅜ inch	1 to ⅜ inch.			
Pounds per square yard	35	35	50		50	35–45	35	35.			
Cost of double application		$2,696.23		$2,326.21	$2,620		$1,983.90	[5] $4,555.80		[6] $3,894.20	
Seal coat:											
Date of application	November, 1927	October, 1929		August, 1929	October, 1930	May, 1926	July, 1929	August, 1929.			
Grade of tar	TC-4	TC-5		TC-4	TC-5	TC-3	TC-4	TC-4.			
Gallons per square yard	¼	⅓		0.3	0.3	0.3	0.3	⅓			
Cover material:											
Type	Limestone	Limestone		Limestone	Limestone	Limestone	Limestone	Limestone.			
Size	1 to ⅜ inch	¾ to ⅜ inch		¾ to ⅜ inch	¾ to ⅜ inch	⅝ to ⅜ inch	1 to ⅜ inch	1 to ⅜ inch.			
Pounds per square yard	25	20			12 to 16	20	20	20.			

RE-TREATMENT

First re-treatment:							
Date	August, 1930					1928	
Grade of tar	TC-5					TC-4	
Gallons per square yard	0.3					0.3	
Cover material							
Type	Limestone					Limestone	
Size	¾ to ⅜ inch					1 to ⅜ inch	
Pounds per square yard	20					20	
Second re-treatment:							
Date						May, 1930	
Grade of tar						TC-5	
Gallons per square yard						⅓	
Cover material							
Type						Limestone	
Size						¾ to ⅜ inch.	
Pounds per square yard						22	

[1] Width of surface treatment 18 feet on all projects.
[2] Priming material used in mixed base.
[3] Second application also served as seal.
[4] Includes cost of seal application.
[5] Includes cost of crushed stone surface course.
[6] Includes cost of crushed stone and screenings surface course.

TABLE 5.—*Typical specification and test values for the priming materials used*

Grade	TC-1		TC-2		TC-3 [1]		TC-4 [2]	
	Specification	Analyses	Specification	Analyses	Specification	Analyses	Specification	Analyses
Specific gravity at 77° F.	1.10 to 1.18	1.13 to 1.17	1.10 to 1.18	1.14 to 1.15	1.10 to 1.18	1.13 to 1.15	1.10 to 1.18	
Water, per cent	2.0	0.5 to 2.0	2.0	1.1 to 1.7	2.0	1.5 to 2.0	2.0	
Free carbon, per cent	2 to 10	2.1 to 6.2	2 to 10	1.6 to 4.5	2 to 10	2.1 to 3.0	2 to 10	
Specific viscosity, Engler, 40° C.	8 to 13	9.3 to 14.8	13 to 18	13.1 to 17.9	18 to 25	18.9 to 24.7	25 to 35	
Total distillate by weight, per cent.								
To 170° C.	5.0	0.3 to 4.2	5.0	1.1 to 1.8	5.0	1.3 to 2.8	5.0	
To 300° C.	40.0	25.8 to 37.4	35.0	23.9 to 29.8	35.0	24.8 to 31.4	40.0	
Softening point of residue,[3] ° C.	[4] 65	50.0 to 64.5	[4] 65	58.0 to 60.0	65	51.5 to 64.0	65	

[1] This grade was used in the mixed base treatment on project 4.
[2] This grade was used in the mixed base treatment on project 5. Test values on TC-4 materials not available.
[3] Cube-in-water method.
[4] Specification limit was 60 prior to 1928 and 65 for 1928 and after.

During cool weather the primer was heated to a temperature of 100° to 150° F. as an aid in obtaining better penetration into the base. During warm weather it was applied at a temperature of 80° to 100° F. Whenever possible, traffic was prevented from passing over the primed base for a period of 24 to 48 hours. In some cases it was necessary to prime half of the base at a time. The lower photograph of Figure 2 is typical of the appearance of a primed base prior to the second application of tar material.

SECOND APPLICATION OF TAR

The next step in the construction of the surface-treated wearing surface is an application of a more viscous tar material, the function of which is to combine with the mineral aggregate cover to produce a thin well-bonded tar mat which will adhere to the primed base, and provide a wear-resistant and waterproof surface.

The material generally used was a hot application tar, heated to a temperature of 160° to 200° F. It was distributed after thorough penetration of the prime, in a single application at the rate of a third to a half gallon per square yard.

A cold application tar was used as a second application on project 13, where the long haul made it more convenient to use this grade of material. On projects 4 and 5 the type of tar used in the construction of the mixed bases was also employed for the second application or seal.

The kinds and amounts of second application hot tars (TH–1) materials, used on the various projects are given in Table 4 and typical specifications and test values in Table 6.

TABLE 6.—*Typical specifications and test values of the hot tar materials used*

	Specification	Analyses
Water	None.	None.
Float test at 32° C., seconds	[1] 90 to 150 [2] 120 to 170	[1] 109 to 131 [2] 121 to 169
Distillation by weight, per cent:		
Total to—		
170° C., not more than	1	0.0 to 0.7
235° C., not more than	10	0.5 to 9.1
270° C., not more than	15	8.2 to 17.4
300° C., not more than	25	12.7 to 23.5
Residue, by weight, per cent, not less than	75	
Specific gravity of total distillate at 25° C., not less than	1.03	
Softening point of residue[3] °C., not more than	75	53.3 to 73.0
Total bitumen soluble in CS_2, per cent	82 to 95	83.9 to 92.7

[1] Specification limits used up to and including 1927.
[2] Specification limits used after 1927.
[3] Cube-in-water method.

APPLICATION OF COVER MATERIAL

Following the second application of tar a cover of aggregate was applied in sufficient quantity to combine with the tar material to form a stable, nonskid, and wear-resistant mat.

The cover material was crushed granite or limestone of various gradations. On the majority of the projects it was 1 to ⅜ inch in size; on others it was 1¼ to ⅜ inch, ¾ to ¼ inch, and ⅝ to ⅜ inch. The amount of cover ranged from 35 to 50 pounds per square yard. Spreading was done by hand or with mechanical spreaders. Hand spotting and broom dragging followed spreading. The surface was then rolled, generally with a 5-ton roller, until the stone was thoroughly seated in the tar. The amount of rolling depended to some extent on the toughness of the cover material. The kind and quantities of cover used are given in Table 4. The following mechanical analyses are typical of the grading of the larger sized cover materials.

Passing 1¼ inch, retained on 1 inch, per cent	11	4
Passing 1 inch, retained on ¾ inch, per cent	63	28
Passing ¾ inch, retained on ½ inch, per cent	23	32
Passing ½ inch, retained on ¼ inch, per cent	2	29
Passing ¼ inch, per cent	1	7
Total	100	100

SEAL TREATMENTS APPLIED AFTER SOME MONTHS OF COMPACTION BY TRAFFIC

A seal treatment is generally applied to the surface at some interval following the original construction in order to further waterproof, bond, and smooth the surface.

A seal treatment was applied, with one exception, to all the sections studied. The interval between the original treatment and the seal varied from three months to a year. A cold-application tar of either 18 to 25, 25 to 35, or 35 to 50 viscosity was used, except on project 8, where a hot-application tar was used. Both granite and limestone were used as cover material. The size was generally ⅜ to ¾ inch, although on a few of the older projects 1 or 1¼ to ⅜ inch material was used. The larger size was used in order to thicken the mat as well as to obtain a more effective early smoothing of the surface. The amounts and kind of material used are given in Table 4. Typical specifications and test results of the cold-application tars used for the second application on project 13 and those used in the seal treatments are given in Table 7.

Some of the older projects have received re-treatments. The materials and quantities used were similar to those used in the seal treatments except that on the more recent work there has been a tendency to use cold-application tar of a higher viscosity. Details of these re-treatments also are given in Table 4.

Mechanical operations common to surface treatment, seal, and re-treatment work are illustrated in Figure 3.

SATISFACTORY SURFACE TREATMENT ON PROJECT 11, OVER CLEAN SAND-GRAVEL ROAD. LOOSE MATERIAL ON UNTREATED ROAD STABILIZED BY HEAVY APPLICATION OF PRIMER

SURFACE TREATMENT BUILT OVER LOOSE SAND-GRAVEL BASE. SURFACE CONDITIONS ARE TYPICAL OF THOSE EXISTING ON THIS TYPE OF ROAD

SAND-CLAY-GRAVEL ROAD FOLLOWING APPLICATION OF THE TAR PRIME COAT PREVIOUS TO APPLYING THE SECOND APPLICATION OF TAR

FIGURE 2.—EXAMPLES OF SURFACE TREATMENT

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TABLE 7.—*Typical specification and test values of the cold application tars used in the second application, seal, and re-treatment*

Grade	TC-3		TC-4		TC-5		TC-6	
Used in	Seal		Seal		Seal and retreatment		Second application [1]	
	Specification	Analyses	Specification	Analyses	Specification	Analyses	Specification	Analyses
Specific gravity at 77° F.	1.10 to 1.18	1.17	1.10 to 1.18	1.13 to 1.16	1.10 to 1.18	1.15 to 1.18	1.10 to 1.18	1.16 to 1.17
Water, per cent	2.0	1.0	2.0	0.9 to 1.9	2.0	0.8 to 1.9	2.0	1.5 to 1.9
Free carbon, per cent	2 to 10		2 to 10	3.9 to 4.3	2 to 10	1.8 to 6.5	2 to 10	
Specific viscosity, Engler, 40° C.	18 to 25		25 to 35	26.1 to 34.3	35 to 50	35.0 to 48.4	50 to 65	56.6 to 62.0
Total distillate by weight, per cent:								
To 170° C.	5.0	1.7	5.0	0.3 to 1.6	5.0	0.3 to 2.0	5.0	0.2
To 300° C.	35.0	35.0	40.0	25.0 to 29.0	40.0	20.8 to 28.0	40.0	21.2 to 21.3
Softening point of residue [2], ° C.	65		65	50.0 to 61.5	65	42.0 to 62.0	65	59.0 to 62.0

[1] Used on project 13 only.

[2] Cube-in-water method.

DISCUSSION OF THE PROJECTS STUDIED

The cost of surface treatment, exclusive of the seal, on the projects studied ranged from about $1,500 to $2,700 per mile of 18-foot width, depending largely upon the availability of materials. Patrol maintenance cost from about $200 to $300 per mile per year, and seal and re-treatments from about $1,050 to $1,250 per mile.

ROUGHNESS INDICATOR USED TO GAGE RIDING QUALITIES OF SURFACES

The surface roughness of the projects as indicated by roughness indicator [3] readings varied considerably, not only between projects but between various portions of the same project. Readings were taken in June and December, 1931, with a roughness indicator installed on a Chevrolet sedan operated with 35 pounds tire pressure and at a speed of 25 miles per hour.

The results given in Table 8 show that lower readings were obtained in December than in June. This difference was probably due in large part to the greater resistance to movement at low temperatures of the hydraulic shock absorbers with which the car was equipped.

TABLE 8.—*Roughness indicator readings taken on the projects studied*

Project	Size of cover material used in seal and re-treatment	Average daily traffic	Average roughness factor per mile	
			June, 1931	December, 1931
	Inches	*Vehicles per day*		
1	¾ to ⅜	620	51	46
2	¾ to ⅜	540	87	54
3	¾ to 3/16	660	86	94
4	1 to ⅜	360	122	80
5	1¼ to ⅜	620	130	
6	¾ to ⅜	360	78	55
7	¾ to ⅜	170	79	59
8	¾ to ¼ / 1¾ to ⅜	[2] 500	102	83
9	1 to ⅜	600	95	68
10	¾ to ⅜	610	67	59
11	1 to ⅜ / 1¾ to ⅜	630	91	73
12	¾ to ⅜	160	79	54
13		100	140	
14	¾ to ⅜	150	159	
15	¾ to ⅜	110	132	
16	⅝ to ⅜ / 1 1 to ⅜	400	128	98
17	1 to ⅜	180	132	
18	1 to ⅜	220	109	130
19	1 to ⅜	220	129	92

[1] Re-treatment.

[2] Estimate.

Experience with the roughness indicator has shown that the small surface irregularities obtained with a coarse stone cover tend to increase the readings, although the riding qualities of the road may not be affected. The instrument is sensitive to very small axle displacements and will measure roughness due to surface texture in addition to that due to the general contour of the road surface. In other words, its readings are a measure of the relative total roughness of the surface tested. Differences in the measurements which may be due to differences in cover material are not necessarily indicative of significant differences in riding quality.

A study of the values given by the roughness indicator on the projects where ¾ to ⅜ inch cover was used in the seal treatments as compared with those on which 1 to ⅜ inch cover was used show considerably greater roughness for the larger aggregate. Six projects on which the smaller sizes aggregate was used gave an average reading per mile of 74 in June and 54 in December as against corresponding averages of 114 and 92 on four projects having the 1 to ⅜ inch aggregate. While this difference may not be due entirely to size of cover material, it is probable that this factor has had some influence.

Readings taken for comparison during February, 1932, on the newly constructed Mount Vernon Memorial Highway, which is considered to have somewhat better than average smoothness, were as follows:

(*a*) Bituminous concrete on clay-bound gravel base, 81.

(*b*) Bituminous concrete on reinforced concrete base, 59.

(*c*) Sheet asphalt on plain cement concrete base, 49.

SERVICE GIVEN BY TREATMENTS GENERALLY SATISFACTORY

The service behavior of the treatment was, in general, satisfactory although there was considerable variation not only between the various projects but between portions of an individual project.

The type of failure occurring most frequently on the projects was a cracking of the mat, usually confined to small local areas. This type of failure occurred over bases and subgrades having high plastic properties and seemed more pronounced on projects in the coastal area, where poorer drainage conditions no doubt tended to increase the plasticity of the subgrade and base materials. Light re-treatments were generally successful in repairing areas where failures of this sort occurred. Figure 4, A is a typical illustration of such a failure.

Detailed discussion of the base and subgrade materials, their effect on the behavior of the surface treatment and a description of tests for their control is given later in this report. (See pp. 14 to 20.)

The value of applying a tar primer previous to surface treatment has been demonstrated by these studies. The major functions of a primer are to bind, stabilize, and waterproof the top portion of the base and to develop a surface to which the superimposed mat will adhere. Examination of samples taken from the road showed that the primed base and the tar-treated mat were very firmly bonded and that the thickness of the wearing surface was increased by the depth of the tar-penetrated base. The depth of penetration varied considerably, because of the different types of base

[3] This device was described in PUBLIC ROADS, vol. 7, No. 7, September, 1926.

material treated, ranging from ⅜-inch on the well bonded topsoil and sand-clay bases to 1 inch or more on the more open gravel bases. The depth of penetration on the latter type of base was limited largely by the amount and consistency of the tar used.

HOT TAR PREFERABLE FOR SECOND APPLICATION

This study indicates that for surface treatments of this type the use of a hot tar for the second application gives better results than are obtained by the use of a cold tar. Under certain conditions there is a tendency for the cold application tar to penetrate into the primed base rather than to remain on the surface. When this occurs the cover stone is either pressed into the softened base or whipped off by traffic. This condition developed on projects 4 and 5 where the bases were of the mixed prime type. The cold tar used as second application material on these two projects penetrated into the mixed base and softened the top portion so that the cover stone was pressed into the base. As a result there was some shoving and loss of cover material on both projects. Figure 4, B illustrates this condition as it occurred on project 5. Laboratory examination of the mat on project 5 showed the top portion of the mixed base to be generally richer than the lower portion. A sample extracted from the base course in one location which is believed to be fairly representative of this project showed 7.4 per cent of bitumen in the top portion as compared with 4.0 per cent in the bottom portion.

On the majority of the projects studied hot tar was used for the second application. The hot tar gave satisfactory surfaces even where the base, as in project 11, was loose and open, or where the base was not so well compacted by traffic as was the condition on project 15. The greater viscosity of the hot tar prevents its penetration into the base and thereby provides a thicker film to hold the cover stone. Its greater adhesiveness adds immediate stability to the surface and little cover material is lost. It gives a more resilient surface than the cold application material. This property is especially desirable in treatments applied to flexible bases and also in regions where climatic conditions are unfavorable.

COLD APPLICATION TAR ADVANTAGEOUS FOR SEAL TREATMENT

The seal treatment is the final step in the construction of surface treatments of the type studied. It serves to close up the open texture and adds life and some additional road metal to the surface. Later re-treatments, similar to the seal treatment, are applied as maintenance measures, in order that the life and riding qualities of the road surface may be retained.

The materials used in the seal treatments and in the re-treatments were essentially of the same type. On the more recent work the tendency was to use a cold application of higher consistency than that of the tars formerly used. On the projects studied, the most viscous tar applied was the TC–5 having a specific viscosity of 35 to 50 at 40° C. Heavier grades, however, are now coming into use, particularly for re-treatment work.

Two important advantages in the use of cold application material over hot tar in these re-treatments are: (1) A more uniform texture of the surface is obtained, and (2) the cover aggregate can be readily dragged to give a smooth surface. Cold application tars also coat the cover material with a thinner film; and this condition tends to reduce subsequent bleeding and the formation of fat spots.

Hot tar was used in the seal and re-treatments on project 8 where the base was particularly plastic. To overcome this condition it was desirable to provide a thicker and more resilient surface than is ordinarily needed. In cases of excessive plasticity the use of hot tar is beneficial, as it prevents the disintegration of the surface by movement in the base course.

ANALYSIS MADE OF SAMPLES TAKEN FROM TEST HOLES

Samples of the surface mats were taken from a large number of test holes on the projects studied. The percentage and consistency of the bitumen extracted, and the grading of the mineral aggregates are given in Table 9. It was possible in many cases to make a clean separation of the tar mat and the tar-primed base; so that the analyses, extractions, and gradings could be made on each portion of the surface treatment. Where no satisfactory separation could be made, the entire thickness of the surface treatment was analyzed as one sample.

In the examination of the separated mats and primed bases, it was found that the percentage of bitumen extracted from the mats varied from 4.3 to 7.8, with an average of 5.8 per cent. That extracted from the primed bases varied from 2.8 to 6.3 per cent with an average of 4.8 per cent. In the cases where the entire thickness of the surface treatment was analyzed as one sample the percentage of extracted bitumen ranged from 3.3 to 6.9 per cent with an average of 4.6 per cent.

CONSISTENCY OF EXTRACTED BITUMENS DETERMINED BY FLOAT TEST AT 50° C.

While the consistencies of the extracted bitumens from the different projects covered a considerable range, it is of interest to note two points: (1) That, as would be expected, the consistency of the bitumen from the tar mats is much higher than that extracted from the primed bases; and (2) that the priming tar has stiffened considerably, as is shown by the float test results on the extracted bitumen. The float test at 50° C. was used as a measure of consistency. The thin fluid priming tar developed a consistency, as measured by this test, of 19 to 88 seconds, with an average of 59 seconds. These figures indicate a definitely higher consistency than that of the original material, which had a specific viscosity of not more than 18 at 40° C. The bitumens extracted from the tar mats had float test values of from 86 to 214, with an average of 134.

It should be recognized that the percentage of extracted bitumen does not give the exact amount of tar present in the road surface at the time of analysis. On extracting a tar aggregate, the free carbon present in the tar is left in the mineral aggregate and the true tar content is always greater than the value given by the extracted bitumen, the magnitude of the error depending on the amount of free carbon in the tar and on the amount of tar which can not be extracted.

The consistency of the extracted bitumen represents the approximate consistency of the carbon-free constituents of the tar existing in the road surface at the time of the survey, except as the consistency may be affected by the slight loss of volatile materials occurring in the extraction process. This bitumen was recovered under a laboratory technique as careful as possible, and it is believed that there has been no undue hardening due to the procedure used. It is felt that the values as reported are reasonably accurate and comparable. The float test determination is well adapted to the testing of tars since it is little influenced by the presence of inert matter such as free carbon. Therefore, it is felt that the float-test values at 50° C. on the extracted

TABLE 9.—*Analysis of the bituminous mats and primed bases*

Identification			Description of sample	Mechanical analysis of the extracted aggregate, in percentage of total sample														Bitumen extracted	Float test at 50° C. of bitumen extracted	Condition [1] of wearing surface December, 1931
				Screen size							Sieve size									
Laboratory No.	Project No.	Hole No.		2 to 1½ inches	1½ to 1¼ inches	1¼ to 1 inch	1 to ¾ inch	¾ to ½ inch	½ to ¼ inch	¼ to No. 10	No. 10 to No. 20	No. 20 to No. 30	No. 30 to No. 40	No. 40 to No. 50	No. 50 to No. 80	No. 80 to No. 100	No. 100 to No. 200	Passing No. 200			
																			Per cent	*Seconds*	
36198	1	35	Mat and primed base				3.5	8.1	14.4	16.7	12.6	6.5	5.6	5.3	6.4	2.7	6.4	7.7	4.1	45	Good.
35315	2	10	Mat					10.2	26.8	16.1	9.9	4.0	5.0	1.4	5.4	2.0	5.1	5.0	6.1	139	Do.
35315	2	10	Primed base					4.9	8.5	12.4	15.6	9.9	10.5	9.7	10.4	3.3	6.6	5.4	2.8	19	
36130	3	13	Mat				11.1	19.1	21.5	16.9	7.9	2.6	3.1	2.1	3.3	1.2	3.1	3.8	4.3	125	Pitted.
36130	3	13	Primed base					2.9	12.1	12.4	16.4	9.7	11.2	7.3	8.2	2.2	6.1	7.0	4.4	45	
35306	4	1	Mat and primed base						6.0	13.2	17.1	6.7	7.7	7.5	13.5	5.4	10.8	8.3	3.8	46	Lean.
35307	4	2	do					1.1	3.1	8.5	18.5	9.8	10.5	9.5	12.6	4.9	9.3	8.4	3.8	42	Spongy and cracked.
35308	4	3	Mat and primed base, upper portion.					1.8	5.4	7.0	13.4	6.6	8.6	8.8	15.0	6.3	13.7	9.8	3.6	34	Good.
35279	4	3	Mat and primed base, lower portion.				2.3	2.0	3.5	6.5	12.0	6.9	8.4	7.7	14.4	6.8	13.7	12.2	3.6	37	
36194	5	34	Mat and primed base, upper portion.							6.0	11.7	11.4	13.8	13.9	15.0	3.7	7.3	6.8	7.4	27	Do.
36195	5	31	Mat and primed base, lower portion.						.7	3.6	12.1	12.2	14.5	15.3	18.4	4.1	7.6	7.5	4.0	21	
35309	6	4	Mat					5.3	29.0	20.6	10.0	3.5	4.3	3.2	5.1	2.0	4.9	6.9	5.8	214	Do.
35309	6	4	Primed base					1.0	3.3	5.5	21.9	10.7	11.8	8.6	10.9	3.2	8.2	12.1	2.8	80	
35310	6	5	Mat					12.2	30.7	23.5	9.0	2.1	2.5	1.8	3.1	1.2	3.0	4.2	6.7	194	Do.
35310	6	5	Primed base				1.1	3.8	3.9	6.8	25.5	9.7	9.5	7.2	8.9	2.2	6.7	9.7	5.0	72	
35311	6	6	Mat					9.4	35.4	24.4	6.9	2.2	2.5	1.9	3.2	1.4	3.3	4.5	4.9	134	Do.
35311	6	6	Primed base					.3	6.5	9.7	17.8	8.5	7.4	7.3	11.0	6.0	8.1	11.8	5.6	88	
35312	7	7	Mat				6.0	8.2	35.8	20.3	5.4	2.0	2.3	1.8	3.0	1.2	2.6	3.6	7.8	93	Do.
35313	7	8	do					2.9	30.9	25.2	10.5	2.9	3.3	2.3	4.1	1.8	3.9	5.2	7.0	104	Do.
35313	7	8	Primed base		3.6	0	3.4	3.2	4.9	6.6	14.6	8.2	10.2	8.6	11.5	4.7	6.4	10.5	3.6	48	
35314	7	9	Mat				9.2	14.7	22.5	18.7	7.6	2.9	2.2	2.1	4.0	1.6	3.9	5.0	5.6	86	Wavy.
35314	7	9	Primed base				1.9	7.9	8.2	9.3	15.0	6.3	4.9	5.1	10.2	3.8	9.8	12.2	5.4	34	
36133	8	14	Mat				5.1	22.6	28.4	12.2	5.2	1.9	2.1	1.8	3.5	1.4	4.3	5.2	6.3	200	Rough and wavy.
36133	8	14	Primed base					1.0	4.8	6.2	15.5	6.1	6.7	6.4	13.9	3.4	13.7	16.0	6.3	61	
36135	9	15	Mat and primed base					8.9	14.2	8.7	10.7	5.4	7.2	6.0	10.5	2.7	8.1	12.7	4.8	107	Good.
36156	10	21	do				4.1	16.6	18.8	12.3	5.4	2.2	3.9	4.3	8.2	3.3	5.7	8.3	6.9	199	Do.
36153	11	23	Mat						8.6	22.2	19.0	15.2	5.0	3.8	6.3	2.4	5.7	7.1	4.7	110	Do.
36153	11	23	Primed base				7.9	4.2	11.0	9.7	12.3	6.1	7.5	6.1	9.7	3.8	8.1	10.2	3.4	68	
36171	12	30	Mat and primed base			3.6	12.6	11.3	12.5	7.7	5.2	4.5	5.0	5.3	8.0	2.5	6.1	7.5	5.2	54	Do.
36140	13	18	do			8.4	6.1	14.1	18.6	12.4	7.4	3.0	3.9	3.4	6.3	2.0	4.9	6.0	3.5	74	Wavy.[2]
36143	14	19	Mat					4.9	32.0	27.5	8.8	1.5	1.9	1.3	2.6	1.5	4.8	7.9	5.3	93	Slightly wavy.[2]
36143	14	19	Primed base				6.7	5.2	16.2	10.5	7.6	3.0	3.7	3.5	8.9	4.5	11.6	14.0	4.6	63	
36146	16	20	Mat					5.0	31.7	25.5	10.7	2.9	2.9	1.9	3.4	1.4	3.2	6.5	4.9	115	Good.
36146	16	20	Primed base					1.6	13.2	14.7	9.8	3.6	5.0	5.4	14.7	4.4	10.7	11.4	5.5	77	
36171	17	29	Mat and primed base			13.7	9.0	9.9	9.5	5.1	6.1	4.6	4.7	5.0	9.6	3.4	7.2	7.8	4.4	163	Do.
36161	18	26	do	20.6	13.8	11.4	6.1	5.0	5.9	3.4	4.2	3.4	4.7	5.3	5.2	1.2	2.8	3.7	3.3	300	Do.
36165	18	27	do				9.7	14.3	18.8	16.2	9.2	3.3	3.5	3.1	5.8	1.6	3.9	5.9	4.7	167	Rutted.
36168	19	28	do					6.9	15.7	16.4	12.5	6.9	6.0	5.3	7.1	2.4	4.9	10.0	5.9	117	Good.

[1] Conditions where samples were taken.

[2] Base corrugations not removed before treating.

bitumens, representing the pure bitumens of the tars, may be considered as comparable to the consistency of the tars in the road surfaces at the time of extraction even though the free carbon is lacking.

NO UNDUE HARDENING OF TAR MATS SHOWN

If the consistencies of the extracted bitumen as given by the float test at 50° C. are compared with the specific viscosity or float test at 32° C. on the material at the time of construction it will be evident that considerable hardening has taken place. While it is no doubt true that the tar on the top surface of the road exposed to light and oxidation may be quite hard, the consistency of the tar extracted throughout the entire mass of the surface mat did not indicate in general any detrimental hardening on those projects which used a hot tar for the second application and a lighter cold tar for seal. In the float test at 50° C. the resultant tar extracted from the wearing suface, not including the primed base, gave results ranging from 86 to 214 seconds. In general the older projects showed higher float test values at 50° C., indicating the effect of longer weathering in hardening the bituminous material. On only one project was the consistency of the extracted bitumen greater than the consistency requirement for tar materials which are generally used in the initial construction of penetration macadam. This fact would seem to indicate that the tar in the mats at the time of the survey was still live and plastic.

CRUSHING OF COVER MATERIAL INDICATED BY ANALYSIS

The gradings of the aggregates extracted from the bituminous mats showed considerable crushing of the cover material to have occurred. It will be observed in Table 9 that the percentages passing the ¼-inch screen and the No. 200 sieve are relatively high, compared with the typical grading of the cover stone used on the majority of the projects.

The preponderance of coarse aggregate on project 18, as shown by the grading of sample No. 36161, may be explained by the fact that on one portion of this project the base was covered with coarse stone before the application of the surface treatment. Sample No. 36165 is the sand-clay portion which did not receive the stone veneer treatment.

On only a few projects was it possible to obtain samples of the mat separated from the primed base. The cases where such samples were obtained offer a means of estimating the extent to which crushing of the cover material occurred. Granite was used on projects 2, 6, and 7; and samples taken from these projects gave percentages passing the ¼-inch screen ranging from 50.0 to 66.2, with an average of 58.7. The two samples taken from projects 14 and 16, on which limestone was used as a cover material, gave percentages of 36.9 and 36.7, respectively. From these limited data it is evident that there was considerable crushing of both materials, although the limestone was apparently more resistent than the granite.

ABOVE: DISTRIBUTOR APPLYING HOT APPLICATION TAR TO THE PRIMED BASE

BELOW: APPLYING MINERAL AGGREGATE WITH MECHANICAL SPREADERS RESULTS IN A MORE UNIFORM DISTRIBUTION THAN BROADCASTING WITH HAND SHOVELS

ABOVE: DRAGGING COVER MATERIAL WITH A WIRE BROOM DRAG FOLLOWING SPREADING OF STONE COVER. THIS OPERATION SHOULD ALWAYS BE USED TO IMPROVE THE RIDING QUALITIES OF THE SURFACE

BELOW: ROLLING THE SURFACE TO PREVENT DISPLACEMENT AND LOSS UNDER TRAFFIC

FIGURE 3.—MECHANICAL OPERATIONS IN SURFACE TREATMENT WORK

There was little indication of detrimental effects due to the crushing of the cover material. It should be noted, however, that the projects studied were of relatively recent construction. It is believed that excessive crushing of cover material will eventually affect the stability of the surface and that, with due regard to economy, tough, durable stone should be used in surface treatment and re-treatment work. Both the granite and the limestone were considered satisfactory as cover materials; and yet, judging by the crushing which occurred, it is obvious that a more resistent stone should be used if a satisfactory surface is to be retained without too frequent re-treatment.

In some cases the cover stone was spread by hand and in others by mechanical spreaders. Under average conditions considerably better results can be obtained by the latter method and, whenever practicable, mechanical spreaders should be required. In order to insure a uniform texture and a smooth riding surface it is essential that hand spotting and brooming be required when hot application tars are used, and that dragging be required in the case of cold applications.

RESULTS OF SOIL TESTS INTERPRETED

The theory of stability of soil mixtures was discussed in the publication Reports on Subgrade Soil Studies,[4] page 38. In this discussion the soil is represented as consisting of large aggregate and mortar, just as are mixed bituminous and concrete paving materials, the function of the large aggregate being to furnish strength and hardness to the road slab and that of the mortar to bind the large aggregate particles into a stable mass.

[4] Reprint of PUBLIC ROADS, vol. 12, Nos. 4, 5, 7, and 8, June, July, September, and October, 1931.

The large aggregate, which is that fraction of the soil retained on the No. 10 sieve, should be hard and tough enough to resist crushing under traffic and can be present in amounts up to at least 50 per cent of the total soil mixture. The soil mortar consists of a clay binder, a silt filler, and a sand-fine aggregate. The character and proportions of these materials must be such that—

(*a*) They have the cohesion required to bind the particles of both coarse and fine aggregate together adequately;

(*b*) They do not shrink in amounts sufficient either to crack the road slab or to cause the occurrence of small fissures between particles of aggregate, which may facilitate the entrance of water into the interior of the slab;

(*c*) They do not swell on wetting an amount sufficient to push the particles of aggregate out of the positions which they have assumed during the consolidation of the surfacing material.

As a matter of fact the same soil colloids which furnish the cohesion also cause volume change and softening in the presence of water. In practice, therefore, the object is to obtain the highest cohesion possible without jeopardizing the integrity of the slab structure by detrimental volume change or softening of the binder clay.

QUALITY OF CLAY BINDERS INDICATED BY PLASTIC AND CAPILLARY PROPERTIES

In untreated roads the binder should be capable of expanding an amount just sufficient to close the surface pores, thus preventing the water from penetrating and softening the interior of the road surface. When the expansion of the binder exceeds that required to close the sand pores, the sand grains are likely to become unseated, reducing the stability of the mixture. When the binder does not expand sufficiently to close the sand pores, water may enter and soften the road surface. It follows that the amount of binder required to produce a stable mixture depends upon the expansion properties of the binder. Binders which are only slightly expansive may be used in an amount sufficient to fill the pores of the sand almost completely. As the expansive properties of the binder become greater, the amount used without danger of unseating the sand grains must of necessity become smaller and smaller.

Of two soils which have an equal tendency to shrink or expand the one having the greater cohesion should be the better binder. Of two soils having equal cohesion, that having the less tendency to shrink or expand should be the better binder, since a greater amount of it can be used than of the more expansive soil.

The facts outlined above may be summarized in the statement that the suitability of a clay binder is dependent on its cohesive, expansive, and shrinkage properties. The cohesion of soils is related to their plasticity; the shrinkage and expansion of soils depend upon the capillary properties. It follows that the character of soil mortars may be determined in the laboratory by constants which throw light on the plastic and capillary properties of the clay binders. These constants are the liquid limit, plastic limit, plasticity index, shrinkage limit, shrinkage ratio, centrifuge moisture equivalent, and field moisture equivalent.

SUBGRADE SOIL CONSTANTS DEFINED

Complete information on the significance of these constants, their use in practice, and their determination has been published in PUBLIC ROADS and is contained in Reports on Subgrade Soil Studies, referred to above.

In the present discussion the description of the soil constants is limited to that required for an understanding of their use in the work covered by this report.

(1) *Liquid limit.*—The liquid limit is defined as the moisture content of the soil at which small shocks will just cause it to flow. It is the maximum moisture content of plastic soils when in the plastic state, and is indicative of the maximum capillary capacity of all soils, plastic or nonplastic. At the liquid limit the cohesion is practically equal to zero.

(2) *Plastic limit.*—The plastic limit is defined as the minimum moisture content at which the soil can exist in the plastic state.

(3) *Plasticity index.*—The plasticity index is the difference between the liquid and plastic limits. It is an expression of the range of moisture contents through which plastic soils remain plastic and consequently cohesive. The plasticity index, in other words, indicates the amount of water, in excess of that represented by the plastic limit, which must be added in order to reduce the cohesion of the soil to practically zero, its state at the liquid limit. It may, therefore, be considered as a qualitative measure of the cohesive property of the soil.

(4) *Shrinkage limit.*—For practical purposes the shrinkage limit is the maximum moisture content of the soil when it has attained the minimum volume during the drying process. It indicates elastic properties of soils, high values indicating high elasticity.

The shrinkage limit indicates minimum porosity produced by maximum capillary pressure due to evaporation. A high shrinkage limit indicates high porosity under maximum capillary pressure. Shrinkage limits of 30 or more are characteristic of elastic soils.

In binders elasticity is not detrimental since the important quality of binders is cohesion and not supporting power. The shrinkage limit, however, has an important bearing on the qualities of binders, because of its relation to the field moisture equivalent. The more nearly the shrinkage limit approaches the field moisture equivalent the less the volume change of the soil under field conditions. A shrinkage limit equal to or greater than the field moisture equivalent indicates no volume change under field conditions.

(5) *Shrinkage ratio.*—When a soil loses moisture by evaporation above the shrinkage limit it shrinks proportionately to the amount of moisture loss. The shrinkage ratio expresses the percentage of soil shrinkage in terms of the percentage of moisture loss. Thus, a shrinkage ratio of 1.8 signifies that above the shrinkage limit a loss of moisture equal to 1 per cent of the weight of the dry soil will cause the soil mass to shrink 1.8 per cent of the volume of the dry soil.

(6) *Centrifuge moisture equivalent.*—The centrifuge moisture equivalent is defined as the percentage of water retained by the soil when compressed by a centrifugal force equal to 1,000 times the force of gravity for one hour.

Generally, except in certain silt soils and in soils which waterlog, the centrifuge moisture equivalent is less than the liquid limit. The centrifuge moisture equivalent indicates the affinity of a soil for water as a result of its capillary properties. The lower the centrifuge moisture equivalent of a soil, the less is its ability

to take up moisture by capillarity. For soils with equal liquid limits, the less the centrifuge moisture equivalent the less is the ability of the wet soil to take up capillary moisture and the lower will be the capillary rise.

Centrifuge moisture equivalents which exceed liquid limits without water-logging indicate extremely high capillarity. Centrifuge moisture equivalents with water-logging indicate impermeability caused by gluey colloids.

Field moisture equivalent.—The field moisture equivalent is defined as the amount of water which a soil will absorb when its moisture content is gradually increased by adding water. As water is added to a soil powder, cohesive or glue-like properties are developed and continue to increase until a certain amount of moisture, depending upon the character of the soil, is absorbed, and a maximum cohesion is developed. With the increasing cohesion the soil grains are more and more restrained from separating. Moisture can enter the soil only by separating the particles; and when the cohesive force exceeds the capillary force tending to cause the separation, no more moisture can enter and the field moisture equivalent is reached. This constant is, therefore, an index of the cohesive properties of the soil.

The addition of moisture in excess of the field moisture equivalent, by means of manipulation, reduces the cohesion until the liquid limit is reached, at which the cohesive properties virtually disappear. The field moisture equivalents of expansive soils which have no cohesion are equal to the liquid limits.

If the liquid limit is greater than the field moisture equivalent the following relations hold true: The greater the difference between field moisture equivalent and liquid limit, the greater is the plasticity of the soil, the greater is the resistance of damp soil to the entrance of capillary moisture, and the less is the tendency of the soil to expand in the presence of moisture.

CONSTANTS OF AVERAGE OR STATISTICAL SOILS USED AS BASIS OF COMPARISON

These constants serve to furnish information on the three important properties of binder clays, their cohesion, their water-retentive or expansive properties, and their shrinkage properties. They do not give quantitative measurements of cohesion, shrinkage, and expansion. Their value lies in the fact that the magnitudes of the constants and their quantitative relations to one another are associated with the characteristics of soil groups whose properties have become known qualitatively through their performance in service.

To facilitate the use of the constants it is necessary to have—

(*a*) A basis of comparison by means of which it is possible to obtain some quantitative conception of the degree to which these three properties are present in a binder clay;

(*b*) A knowledge of the relations between soil performance and the presence of these properties in different degrees.

The basis used for comparison is the set of relationships which was found to exist between the average clay contents and the average soil constants of more than 5,000 soil samples. An "average" or "statistical" soil may be defined as a hypothetical soil having constants conforming to these averages. Table 10 gives the soil constants of statistical soils for varying percentages of clay.

TABLE 10.—*Soil constants of statistical soils*

Percentage of clay	Liquid limit	Plasticity index	Shrinkage limit	Moisture equivalent	
				Centrifuge	Field
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*
1	13	0	17	9	9
2	14	0	17	10	10
3	14	0	17	10	10
4	15	1	17	11	11
5	15	1	17	11	11
10	18	3	16	13	14
15	23	6	16	17	19
20	28	9	15	20	22
25	35	13	15	25	26
30	42	18	14	30	29
35	49	22	14	35	31
40	56	26	13	40	34
45	62	30	13	45	35
50	69	34	12	50	37
55	76	39	12	55	39
60	83	43	12	60	41
65	90	48	11	65	42
70	97	52	11	70	44
75	104	56	11	75	46
80	111	61	10	80	47
85	118	65	10	85	48
90	125	69	10	90	50
95	132	74	9	95	51
100	139	78	9	100	52

In this table we find that the average or statistical soil having a clay content of 50 per cent has physical constants as follow:

Liquid limit	69
Plasticity index	34
Shrinkage limit	12
Shrinkage ratio [5]	2
Centrifuge moisture equivalent	50
Field moisture equivalent	37

Comparisons may be made with soils having the same clay content but different constants. A soil with a similar clay content but with a higher plasticity index, say 45, is suggestive of clay more active than the average with respect to plasticity, volume change, etc., the higher plastic properties being due mainly to the presence of gluey colloids. Likewise, a soil with a similar clay content but with a lower centrifuge moisture equivalent, say 40, suggests a clay of lower water retentive properties than the average.

STATISTICAL RATIOS USED TO EXPRESS RELATION OF ACTUAL TO STATISTICAL SOILS

It is convenient in practice to express the relationships of constants of a given soil to those of the average soil as a ratio termed the average or statistical ratio. Thus the average ratio of the plasticity index of 45, referred to above, is $\frac{45}{34}$, or 1.32, and the average ratio of the centrifuge moisture equivalent of 40, also referred to above, is $\frac{40}{50}=0.80$.

In the case of natural soils, in which the active portion of the soil tested comprises a large part of the entire soil sample, the soil constants as such are usually capable of indicating clearly the soil characteristics. In graded mixtures, however, the clay binder may be but a small fraction of the portion of the soil tested. In such cases the use of the ratios to supplement the constants may be required in order to disclose the character of the clay.

Without these average ratios it would be necessary to perform the very painstaking and costly operations of separating the clay contents from the soil samples

[5] The average shrinkage ratio is obtained by substituting the average specific gravity of soils, 2.65, and the statistical shrinkage limit (Table 10) in the formula:

$$\text{Shrinkage ratio}=\frac{100\times\text{specific gravity}}{100+\text{shrinkage limit}\times\text{specific gravity}}$$

and testing them separately in order to determine the character of the clay. Since a knowledge of the mechanical analysis without a knowledge of the character of the clay binder is inadequate to determine the efficiency of the soil for use either as road surfaces or as bases for surface-treated roads, the importance of the average ratios will be readily understood.

GROUP CLASSIFICATION OF SOILS SERVES AS KEY TO PERFORMANCE

Because of the fact that the presence of certain soil constituents indicates the important soil properties, it has been found possible to arrange the various soils in groups which are indicative of their performance. The characteristics of the uniform soil groups are discussed in the following paragraphs.[6]

Group A–1.—Well-graded material, coarse and fine, excellent binder. Highly stable under wheel loads, irrespective of moisture conditions. Functions satisfactorily when surface-treated or when used as a base for a relatively thin wearing course.

Group A–2.—Coarse and fine materials, improper grading or inferior binder. Highly stable when fairly dry. The more plastic soils of this group are likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long continued dry weather.

Group A–3.—Coarse material only, no binder. Lacks stability under wheel loads but is unaffected by moisture conditions. Not likely to heave because of frost or to shrink or expand in appreciable amount. Furnishes excellent support for flexible surfaces and also for relatively thin rigid pavements.

Group A–4.—Silt soil without coarse material, and with no appreciable amount of sticky colloidal clay. Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon removal of load. Likely to cause cracking in rigid pavements as a result of frost heaving, and failure in flexible pavements because of low supporting value.

Group A–5.—Similar to Group A–4, but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadam during construction and with retention of good bond afterwards.

Group A–6.—Clay soil without coarse material. In stiff or soft plastic state absorbs additional water only if manipulated. May then change to liquid state and work up into the interstices of macadams or cause failure due to sliding in high fills. Furnishes firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking in rigid pavements.

Group A–7.—Similar to Group A–6, but at certain moisture contents deforms quickly under load and rebounds appreciably upon removal of load, as do soils of Group A–5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in the case of Group A–6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterwards. May contain lime or associated chemicals productive of flocculation in soils.

As the characteristics of the mortar change from those of the A–1 subgrade, the mortar may become an A–2 material of one of several varieties. Increase in sand content, for instance, would produce a friable variety approaching the A–3 material; increase in clay content would produce a plastic variety approaching the A–6 or A–7 materials; and increase in the silt content would produce a highly capillary variety approaching the A–4 or A–5 material.

Without surface treatment the A–1 material should prove satisfactory on both wet and dry subgrades; the plastic A–2 may serve well on dry subgrades; the friable fairly well on wet subgrades; and the capillary variety is likely to fail in either case. The cohesionless A–3 and the highly elastic A–5 soils are unsuited for road surfaces. The A–4, the A–6, and the A–7 soils are likely to be satisfactory as untreated road surfaces only in dry weather.

EFFECTS OF SURFACE TREATMENTS DISCUSSED

Bituminous surface treatments provide (*a*) cohesion which serves to bind the top particles of the base together, (*b*) a mat which resists abrasion and protects the base from the suction, thrust, or other shocks produced by traffic, and (*c*) an impervious top which prevents the entrance of surface water into the base and the evaporation of capillary moisture from the base.

The wear-resistant mat and the cohesive action of the bituminous surface treatment are beneficial for bases comprised of all groups of soils.

The ability of surface treatments to prevent surface water from entering the soil bases is especially advantageous to those group A–6 and A–7 soils which are so impervious that capillary moisture can not enter from below in detrimental amounts. Under such conditions, these soils make excellent bases for surface treatments.

The increase of capillary moisture in the soil slab due to the prevention of evaporation may have several important effects: (*a*) It may soften the more plastic varieties of clay binder and thus make unstable the soil slabs which contain the amounts of this type of clay required for stable untreated surfaces; (*b*) it may provide additional cohesion for holding together the grains of over-sanded mixtures which become very loose in dry weather when untreated; (*c*) it may cause loss of stability in bases composed of the highly capillary varieties of A–4 soils.

It is thus evident that the amount of clay necessary to bind an untreated road may be excessive if the road is treated, the rise of capillary moisture causing the base to soften and fail. The occurrence of failures over plastic bases and subgrades on some of the projects covered by this study is attributable to this phenomenon.

The present tendency is to use less clay in new construction and, on previously untreated roads to modify the plasticity of the mortar by the addition of granular material or by some other suitable means, prior to application of the surface treatment.

Materials belonging to the various soil groups may be used successfully as bases for surface treatment under the following conditions: A–1 materials on dry or moderately wet subgrades; plastic and capillary A–2 materials on dry subgrades and friable A–2 materials on both wet and dry subgrades; well drained A–4, materials not subjected to frost action and certain impervious varieties of the A–6 and A–7 subgrade materials. A–1 materials on very wet subgrades and A–2 plastic and capillary materials on either very wet or moderately wet subgrades are likely to prove troublesome because of softening of the clay binder.

[6] See Reports on Subgrade Soil Studies, pp. 17 to 49.

Because it increases the capillary moisture content, and because this in turn produces additional cohesion, a surface treatment may not only stabilize the surface of a soil road, but may also, in the case of materials having but little cohesion, exercise a stablizing benefit throughout the entire depth of the road base and a part of the subgrade beneath. This fact explains why many materials, not suited for soil roads without treatment, perform satisfactorily when surface-treated.

The grading of the soils which contained kaolin as the clay binder and which were found to furnish highly stable untreated road surfaces in Georgia was as follows:

Material retained on the No. 10 sieve, not more than about 50 per cent; the soil mortar, that fraction passing the No. 10 sieve, consisting of clay, 5 to 10 per cent; silt, 10 to 20 per cent; total sand, 70 to 85 per cent; and coarse sand, 15 to 60 per cent. Average effective size approximately 0.01 millimeter and uniformity coefficient greater than 15.

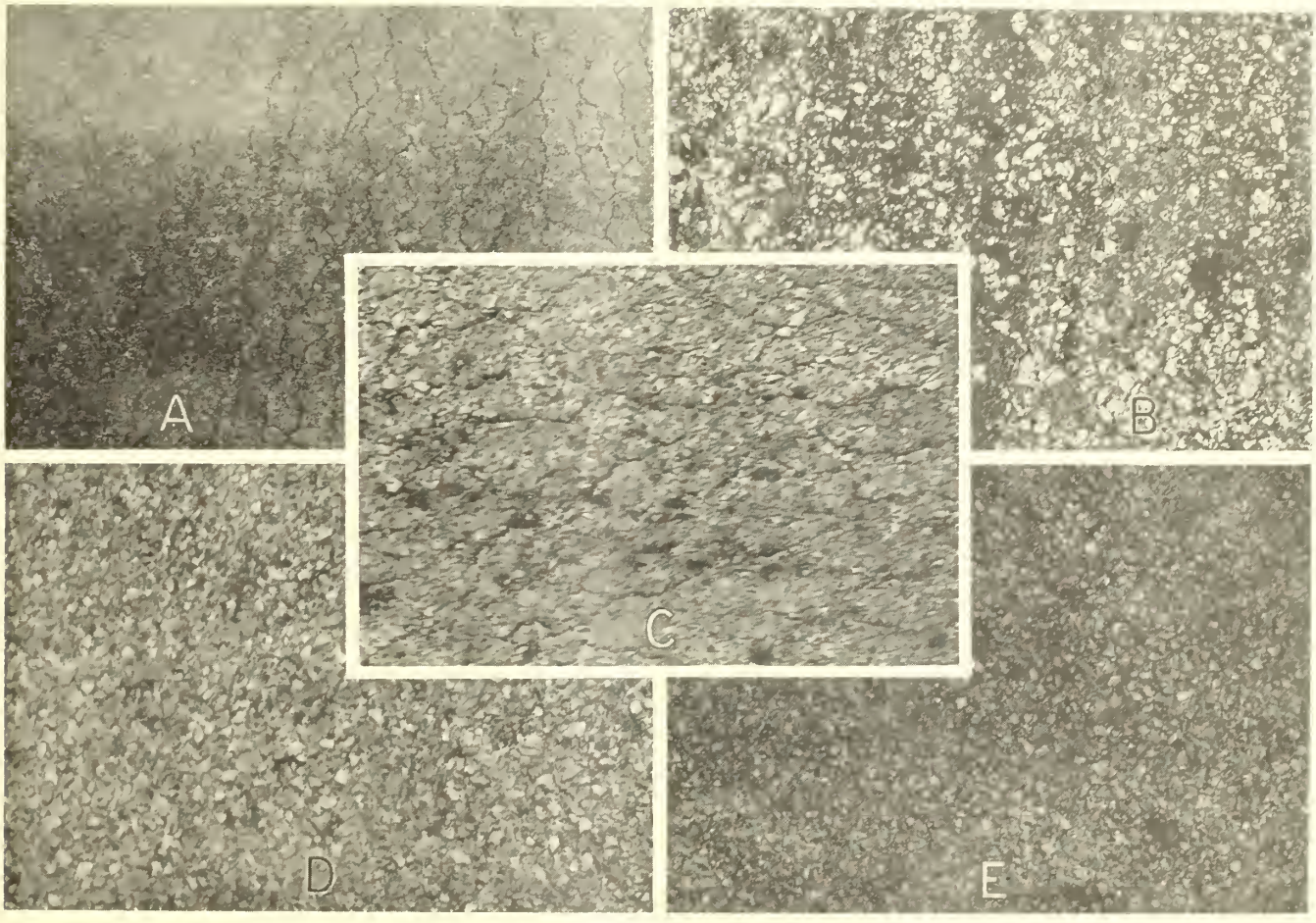

FIGURE 4.—EXAMPLES OF SURFACE TEXTURE ON TAR-TREATED PROJECTS

A, CHARACTERISTIC FAILURE ON SAND-CLAY-GRAVEL BASE OF HIGH PLASTICITY UNDER UNFAVORABLE DRAINAGE CONDITIONS

B, SURFACE CONDITION TYPICAL OF THE TWO MIXED BASE PROJECTS, 4 AND 5, WHERE COLD TAR MATERIALS TC–3 AND 4 WERE USED FOR SECOND APPLICATION. PHOTO SHOWS COVER STONE EMBEDDED IN THE SOFTENED PRIMED BASE RATHER THAN BEING HELD BY A TAR FILM

C AND D, PROJECT 8, ILLUSTRATING THE EFFECT OF MICA IN THE BASE. BASE MATERIALS IDENTICAL IN CHARACTER EXCEPT FOR THE MICA PRESENT. FIGURE C REPRESENTS THE SURFACE WHOSE BASE CONTAINED AN APPRECIABLE AMOUNT OF MICA AND FIGURE D THE SURFACE WHOSE BASE WAS FREE FROM MICA

E, PROJECT 7, HOLE 7. GOOD SURFACE CONDITIONS OVER A–7 SOIL UNDER FAVORABLE DRAINAGE CONDITIONS

KAOLIN REPRESENTATIVE OF GOOD BINDER CLAYS

It will be readily understood that the two important requirements of binder clay, to wit, appreciable cohesion and no detrimental volume change, are the requirements also of good pottery clay. The cohesion is required to hold the molded pieces together during the burning and the absence of volume change is required to prevent cracking of the heated pieces. It is, therefore, not surprising to learn that kaolin (hydrous silicate of aluminum), an excellent China or porcelain clay, seems to satisfy both the practical and theoretical requirements of good binder clay, according to experience with untreated topsoil roads in Georgia.

The character of kaolin when compared with that of other soils is disclosed by its statistical ratios, which are as follows:

Liquid limit	1.07
Plasticity index	1.00
Shrinkage limit	2.77
Centrifuge moisture equivalent	1.23
Field moisture equivalent	1.06

Table 11, which contains the ratios of a limited number of typical soils, assists in revealing the significance of the ratios of kaolin binders. From these values it can be seen that the liquid limit of kaolin, like that of the A–6 and A–7 clays, is equal approximately to that of the statistical soils. It is apparent that kaolin will

TABLE 11.—*Ratios of test constants of typical soils to those of statistical soils*

Soil	Ratio to statistical soil					Subgrade group
	Liquid limit	Plasticity index	Shrinkage limit	Moisture equivalent		
				Centrifuge	Field	
Florida sand	0.73	-------	0.94	0.45	1.45	A-3
California sand	1.43	-------	1.00	.30	1.70	A-3
New Hampshire silt	.76	-------	1.67	.79	.96	A-4
Missouri silt	.84	0.67	1.71	1.07	1.04	A-4
Oregon silt	1.82	1.60	2.06	2.25	2.06	A-5
Maryland silt	2.71	2.21	6.71	2.94	3.57	A-5
Virginia clay	1.08	1.51	1.10	2.04	.67	A-6
Texas clay	1.12	1.48	.92	1.88	.81	A-6
Minnesota gumbo	1.05	1.31	1.31	1.40	1.31	A-7
Mississippi clay	1.00	1.34	.90	1.43	1.11	A-7

not lose stability until it takes up as much moisture as that required to soften the fine-grained cohesive soils.

The typical silts listed in Table 11 have plasticity index ratios less than the liquid limit ratios, while the clays have plasticity index ratios greater than the liquid limit ratios. This fact indicates that clays have relatively much greater cohesion than silts. The plasticity index ratio of kaolin, however, just about equals the liquid limit ratio. It follows that the cohesive properties of kaolin are somewhat greater than those of silts but not as great as those of the clays and gumbos.

With this moderately high cohesion, kaolin has an exceptionally high shrinkage limit ratio which, combined with the low field moisture equivalent ratio, suggests negligible shrinkage properties. Its centrifuge ratio of 1.23 indicates water retentive properties smaller than those of the A–5, A–6, and A–7 soils.

STATISTICAL RATIOS EXPLAIN PERFORMANCE OF MATERIALS USED ON PROJECTS

Tables 12 and 13 give the statistical ratios of the materials used as subgrades and bases on the various projects studied. The clay contents used in these tables are based upon that portion of the sample passing the No. 40 sieve, which is the portion used in determining the soil constants. These clay contents are important only in determining the statistical ratios, and should not be confused with those given in Tables 2 and 3, which are the true clay contents of the soil mortar (material passing the No. 10 sieve).

An examination of Tables 12 and 13 will help to explain the behavior of the surface-treated roads observed in the survey. It should be borne in mind that the properties of good binders are represented by those of kaolin; and that bituminous surface treatments permit a much wider range in soil properties than is practicable in untreated roads.

According to these tables seven base samples have relatively high plasticity index ratios (greater than 1, that of kaolin). They are 6369 and 6206, project 1; 6064 and 6063, project 2; 6195, project 3; 6062, project 7; and 6197, project 14.

With the high plasticity indices, 5 of these samples, 6369, 6064, 6195, 6206, and 6063, have field moisture equivalent ratios less than 1.06, that of kaolin. Three of these 5 samples, 6369, 6064, and 6195, were taken from portions of roads which had developed defects. The clay contents[7] of samples 6369 and 6064, were, respectively, 21 and 22 per cent, while the clay content of the third sample, 6195, was 13 per cent. The surface defect in project 3 (base sample 6195) is pitting, and this may be due to the character of the surface treatment rather than to the characteristics of the base material. The clay contents of the 2 surfaces representative of good results—samples 6063 and 6206—were, respectively, 14 and 15 per cent.

Samples 6062 and 6197, which also had high plasticity indices, were taken from sections reported as having wavy surfaces. Field moisture equivalent ratios greater than 1.06, that of kaolin, indicate that these soils have relatively high moisture absorptive properties.

[7] Clay contents of soil mortar, that fraction passing No. 10 sieve, Tables 2 and 3.

TABLE 12.—*Ratios of test constants of soils tested to those of statistical soils having the same percentage of clay on projects having sand-clay-gravel base*

Project No.	Laboratory No.	Hole No.	Course	Ratios to statistical soil having same percentage of clay: Clay content[1]	Liquid limit ratio	Plastic index ratio	Shrinkage limit ratio	Centrifuge moisture equivalent ratio	Field moisture equivalent ratio	Condition of wearing surface
				Per cent						
1	S-6369	36	Base	26	0.94	1.36	1.20	0.65	0.88	Badly cracked.
	S-6206	35	do	23	1.00	1.36	1.33	.78	1.00	Good.
	S-6194	35	Subgrade	15	.74	.50	[2] 94	1.00	.84	
2	S-6063	10	Base	22	.97	1.30	1.27	.86	1.04	Do.
	S-6057	10	Subgrade	16	.79	0	1.13	.47	.90	
	S-6064	12	Base	27	.89	1.20	1.21	.67	.81	Cracked.
	S-6059	12	Subgrade	12	.95	0	[2] 1.31	.50	1.25	
7	S-6060	7	Base	46	.75	.90	1.38	.65	.75	Good.
	S-6054	7	Subgrade	49	.90	.85	2.58	.69	1.14	
	S-6061	8	Base	20	.43	.22	[2] 1.93	.60	.50	Do.
	S-6055	8	Subgrade	44	.75	.83	1.23	.77	.91	
	S-6062	9	Base	11	.95	1.33	1.13	.79	1.13	Wavy.
	S-6056	9	Subgrade	27	.76	.93	1.14	.67	.78	
11	S-6203	23	Base	6	1.31	0	[2] 1.65	.83	2.08	Good.
	S-6173	23	Subgrade	32	.75	.79	1.50	.88	.83	
13	S-6196	18	Base	8	1.35	0	[2] 1.71	.58	2.15	Wavy, base corrugations not removed before treatment.
	S-6170	18	Subgrade	19	1.04	.75	1.60	1.42	1.41	
14	S-6197	19	Base	12	1.25	1.75	[2] 1.50	1.00	1.50	Slightly wavy, base corrugations not removed before treatment.
	S-6171	19	Subgrade	31	1.02	.61	2.36	1.19	1.38	
15	S-6199	21	Base	10	1.44	0	----------	1.69	2.00	Good.
	S-6200	21	Subgrade	21	.90	.78	1.53	1.10	1.09	
16	S-6198	20	Base	13	.90	0	[2] 1.44	.87	1.12	Do.
	S-6172	20	Subgrade	31	1.00	1.06	1.86	1.19	1.14	
	S-6201	22	Base	10	1.22	0	[2] 1.50	.77	1.50	Slightly cracked.
	S-6202	22	Subgrade	11	1.32	0	[2] 1.81	1.21	1.80	

[1] Based on mechanical analysis of material passing No. 40 sieve.
[2] Soils which have no significant shrinkage limit. Shrinkage limit computed for purpose of comparison.

TABLE 13.—*Ratios of test constants of soils tested to those of statistical soils having the same percentage of clay on projects having topsoil or sand-clay base*

TOPSOIL BASE PROJECTS

Identification				Ratios to statistical soil having same percentage of clay						
Project No.	Laboratory No.	Hole No.	Course	Clay content [1]	Liquid limit ratio	Plastic index ratio	Shrinkage limit ratio	Centrifuge moisture equivalent ratio	Field moisture equivalent ratio	Condition of wearing surface
				Per cent						
4	S-6043	1	Base	12	0.80	0	[2] 1.25	0.36	1.12	Lean.
	S-6044	1	Subgrade	29	.70	.81	1.21	.72	.71	
	S-6045	2	Base	17	.64	.43	1.00	.78	.75	Spongy and cracked.
	S-6046	2	Subgrade	40	.88	.85	2.08	1.00	1.06	
	S-6047	3	Base	11	.74	0	[2] 1.19	.50	1.00	Good.
	S-6048	3	Subgrade	21	.72	.89	1.20	.71	.78	
6	S-6049	4	Base	22	.50	.50	.87	.50	.57	Do.
	S-6050	5	do	28	.51	.50	1.21	.64	.57	Do.
	S-6051	5	Subgrade	29	1.05	1.00	2.07	1.21	1.21	
	S-6052	6	Base	23	.41	.18	.93	.35	.50	Do.
	S-6053	6	Subgrade	39	.87	.80	2.15	.90	1.09	
8	S-6165	14	Base	17	.76	.57	1.06	1.00	.95	Rough and wavy.
	S-6190	33	do	23	.63	.45	1.27	.78	.79	Badly cracked.
	S-6205	33	Subgrade	25	.77	.85	1.67	.92	.81	
	S-6191	33	do	29	1.18	1.44	1.64	1.31	1.21	
9	S-6166	15	Base	17	.56	.43	[2] .94	.78	.70	Good.
	S-6167	15	Subgrade	48	.78	.82	1.58	.77	1.05	
	S-6168	16	Base	19	.67	.50	1.13	.68	.73	Slightly cracked.
	S-6169	16	Subgrade	34	.87	1.00	1.36	.85	.90	
10	S-6174	24	Base	20	.86	.89	1.27	1.00	.95	Good.
	S-6175	24	Subgrade	21	1.59	1.44	2.67	1.81	1.96	
	S-6176	25	Base	26	.94	.86	1.67	1.04	1.12	Cracked.
	S-6204	25	Subgrade	19	1.56	1.50	2.40	1.32	1.86	
12	S-6186	30	Base	19	.70	.50	1.00	.89	.86	Good.
	S-6188	30	Subgrade	34	.98	.90	2.07	.82	1.16	
	S-6187	31	Base	30	.93	.89	1.64	.93	1.03	Slightly cracked.
	S-6189	31	Subgrade	14	2.41	0	2.94	2.13	3.06	
17	S-6184	29	Base	19	.67	.75	1.07	.89	.73	Good.
	S-6185	29	Subgrade	45	.85	.80	1.92	.73	1.03	
18	S-6180	27	Base	29	.73	.63	1.50	.97	.86	Rutted.
	S-6181	27	Subgrade	19	1.48	1.50	2.40	1.26	1.77	
19	S-6182	28	Base	30	.64	.56	1.21	.77	.69	Good.
	S-6183	28	Subgrade	32	1.27	1.16	2.29	1.25	1.53	

SAND-CLAY BASE PROJECTS

				Per cent						
2	S-6058	11	Base	16	0.75	0	[2] 1.31	0.47	0.90	Good.
3	S-6195	13	do	19	.85	1.25	1.07	.68	.82	Pitted.
	S-6164	13	Subgrade	11	.84	0	1.25	.86	.80	
5	S-6192	34	Base	38	.74	.75	1.77	.63	.85	Good.
	S-6193	34	Subgrade	15	.65	0	[2] 1.13	.29	.89	
18	S-6177	26	Base	19	.85	.88	1.27	1.05	.95	Do.
	S-6178	26	Subgrade	34	.98	1.05	1.86	.91	1.06	
	S-6179	26	do	15	1.48	0	[2] 2.38	1.12	1.89	

[1] Based on mechanical analysis of material passing No. 40 sieve.
[2] Soils which have no significant shrinkage limit. Shrinkage limit computed for purposes of comparison.

Their clay contents were, respectively, 9 and 11 per cent.

The high field moisture equivalent ratios (greater than 1.06) of base samples 6203, project 11; 6196, project 13; 6199, project 15; 6201 and 6198, project 16; and 6043, project 4, combined with plasticity indices of zero suggest the presence of detrimental expansive properties due in part to the absence of cohesion. These samples have clay contents of 11 per cent or less. However, three of the six surfaces represented by these samples are reported as being in good condition.

Seven subgrade samples (Nos. 6054, project 7; 6171, project 14; 6175 and 6204, project 10; 6189, project 12; 6181, project 18; and 6183, project 19) listed as A–5 soils, indicated potential elastic properties high enough to cause their shrinkage limits to exceed 30.

In the case of samples 6183 and 6054 the relatively high plasticity indices of 22 and 29, respectively, indicate cohesive properties tending to reduce the elasticity of the material in service. It is interesting to note that the road surfaces laid on the subgrades from which these two samples were taken remained in good condition. The road surfaces laid on four of the five subgrades represented by the other A–5 soil samples referred to developed defects.

There is nothing in the test results on base samples 6045, project 4; 6165 and 6190, project 8 and 6168, project 9, to account for the defects developed by the surfaces represented by these samples.

The data regarding the desirable properties of soil mortars and subgrade soils is not comprehensive enough to permit the drawing of hard and fast specifications. Added to the information which has been previously collected regarding these matters, the preceding discussion becomes quite indicative. The following is suggested for bases to be surface treated:

1. Binder clays having high plasticity, indicated by plasticity index ratios greater than 1, with normal expansive properties, indicated by field moisture equivalent ratios less than 1.06, should not be present in base mortars in amounts exceeding 20 per cent.

2. Highly expansive clay binders indicated by field moisture equivalent ratios greater than 1.06 are likely to prove troublesome, whether the plasticity is high or low.

3. The less plastic subgrades of the A–5 groups are likely to prove troublesome in surface-treated roads.

4. Binder clays having plasticity index ratios of 1.00 or less and field moisture equivalent ratios not exceeding 1.06 can be used in varying amounts successfully.

For example, as is shown in Table 2, sand-clay-gravels with clay up to 15 per cent functioned well with surface treatments, whereas those on which the treatments cracked contained slightly more than 20 per cent.

The excellent performance of the base represented by sample 6060, project 7, with 36 per cent of clay, may have been due more to good drainage conditions than to good grading of soil material. In this connection, however, it should be recalled that when enough fine material is present to make A–7 soils impervious, they lose stability primarily as a result of water entering from the top. Consequently, as illustrated by the Minnesota gumbos, they are stabilized by surface treatments preventing the entrance of water from above. It is, therefore, possible that soils with either a very large or very small amount of clay may function well when surface-treated, whereas, a soil containing a moderate amount may cause softening and loss of stability.

A number of the sand-clay-gravel base projects were built of river-washed material containing only from 4 to 13 per cent of clay. The surfaces on these projects were generally in better condition than those built on bases having considerably larger percentages of clay binder. The river-bed sand-clay-gravel contained a high percentage of material larger than 2 millimeters, as much as 71 per cent in one instances, and exceeding 50 per cent in 4 cases.

The sand-clay and topsoil bases which are shown in Table 3 to have given good service had clay contents varying from 9 to 26 per cent. The clay contents of the faulty bases varied from 12 to 26 per cent. It is thus apparent that the clay content alone is not a criterion of the service value of the base materials.

PROPERTIES OF SUBGRADE AND BASE MATERIALS IMPORTANT IN SURFACE TREATMENT

It is evident from the foregoing discussion that the behavior of surface treatments depends upon the character of the base and subgrade materials. The suitability of the underlying material is largely dependent upon the character and quantity of the binder. The suitability of an untreated road as a base, or new base construction for surface treatment, is often indicated by its behavior under traffic. An untreated surface which tends to ravel in dry weather will probably be more satisfactory after treatment than one which remains well bonded. This is contrary to early opinions on this subject; but experience has shown that where a given clay is used as a binder a less amount is desirable in bases for surface treatment than is required for an unsurfaced road of the same material. The clay required for stability in the latter tends to become plastic after surface treatment, because the surface mat prevents evaporation and causes the base to yield under traffic; and this leads to weakening and failure of the mat.

The maximum amount of clay binder permissible in the base depends largely upon the fine materials and particularly on the character of the clay itself. Because of the presence in base materials of certain soil constituents which determine to a large extent the behavior of the base, it is possible to apply soil analysis not only to the subgrade soils but to the base course material as well.

Unsatisfactory base materials were used on several of the projects studied, with resulting failure of the surface. In some cases also the constructed base courses were composed of materials inferior to the natural soil in the road. Analyses of the materials before construction would not only have prevented the use of this unsatisfactory material but would have brought out the possibility of using the natural soil or would have indicated the best method of treating it to make it satisfactory.

For example, sand admixtures are likely to prove beneficial to base materials when highly plastic clay with a plasticity index ratio greater than 1.00 is present in amounts exceeding 20 per cent. This same type of clay, in moderate amounts, may prove beneficial in stabilizing elastic bases which have little or no plasticity and have field moisture equivalent ratios greater than 1.00 The same type of clay may be advantageously used in considerable amounts to stabilize highly elastic A–5 soils.

ADAPTABILITY OF VARIOUS SOIL GROUPS FOR SURFACE TREATMENT SUMMARIZED

The performance which may in general be expected from soils of the various uniform groups, when used as bases for surface treatment, is outlined in the following paragraphs:

Group A–1.—Materials would be highly satisfactory but are seldom encountered. There is also the possibility of softening on very wet subgrades.

Group A–2.—Materials range from friable to plastic. The less plastic materials are satisfactory. The more plastic materials are unsatisfactory, except under dry conditions, but are often used. The presence of micaceous materials tends to decrease greatly the stability of the base material. Some suitable treatment for reducing the plasticity of the material or stabilizing bases having the more plastic characteristics should be applied. A large percentage of the bases studied come under this group.

Group A–3.—Materials are highly satisfactory for surface treatment. In the construction of a bituminous mat on this material a large amount of primer will probably be necessary to stabilize the surface of the base. The sand-gravel materials obtained from runs and river beds and used on some of the projects, such as project 11 and parts of projects 13 and 16, are of this group, as are the topsoils used on project 4 and part of project 6.

Group A–4.—Material is not satisfactory for surface treatment except under favorable moisture conditions. Since material is deficient in sand, stabilizing by mixing a fairly light tar with it, or laying a thin surface cover of coarse graded aggregate, would probably be more practicable than adding a large amount of sand or other fine granular materials, which would be required as an admixture. The topsoil on part of project 10 is of this class. This material was not given any special treatment and the surface mat has developed some cracks.

Group A–5.—Materials may not be satisfactory for surface treatment because of detrimental elasticity. Stabilizing by mixing a light tar with the surface would probably be as satisfactory as employing a thick top course of stone or other suitable material.

Groups A–6 and A–7.—Materials generally lose stability as a result of water entering from above. Surface treatments which prevent this entrance of moisture produce fairly satisfactory results. A thin top course of sand added to the surface would greatly improve the material for surface treatment. The A–7 soil, found extensively in the Piedmont area, is a light red-colored material. The base material on part of project 7 is of this type.

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TRANSPORTATION SURVEY REPORTS

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Report of a Survey of Transportation on the State Highways of Vermont. (1927.)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927.)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928.)

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

JANUARY 31, 1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,350.9	$ 4,648,723.85	$ 2,324,361.84	27	150.5	77.0	227.5	$ 893,223.37	$ 446,611.66	10.3	28.1	38.4	$ 3,315,399.04	Alabama
Arizona	1,213.9	3,173,325.89	1,374,568.02	65	125.5	154.5	280.0	9,644.06	1,928.81				128,133.66	Arizona
Arkansas	1,953.7	3,421,030.81	1,612,737.68	62	91.9	40.3	132.2	1,080,612.73	540,119.86	55.5		55.5	1,448,409.36	Arkansas
California	2,459.2	8,278,470.13	2,486,231.74	68	135.6	41.3	176.9	2,193,130.53	201,821.92	70.9		70.9	52,050.49	California
Colorado	1,781.3	2,479,420.76	1,117,579.88	66	100.9	22.7	123.6	712,809.12	320,764.04	46.6		46.6	805,464.56	Colorado
Connecticut	289.8	4,775,965.40	1,870,572.75	63	51.5		51.5	274,024.48	84,250.20	4.5		4.5	17,881.35	Connecticut
Delaware	374.4	538,564.15	101,900.00	55	14.3	17.2	31.5	218,276.10	41,300.00	9.8		9.8	33,025.44	Delaware
Florida	661.0	5,863,099.75	2,795,408.63	54	157.7		157.7	297,518.00	148,758.99	10.6		10.6	1,241,804.16	Florida
Georgia	3,176.8	5,624,315.89	2,276,808.32	57	161.9	174.5	336.4	500,491.90	219,482.10	22.5	26.9	49.4	84,696.48	Georgia
Idaho	1,539.0	2,415,174.17	983,277.00	72	121.7	122.5	244.2	598,446.86	185,127.72	16.5	16.7	33.2	75,427.89	Idaho
Illinois	3,069.7	20,636,314.07	8,115,405.14	79	636.0	33.0	669.0	1,498,259.44	471,696.98	62.4		62.4	126,899.36	Illinois
Indiana	2,004.5	8,514,308.55	3,931,048.24	91	319.8	3.0	322.8	1,725,082.44	184,013.84	87.4		87.4	123,082.17	Indiana
Iowa	3,519.1	5,384,285.39	993,885.38	91	261.1	40.0	301.1	359,559.81	23,060.00	25.0	1.2	26.2	11,498.31	Iowa
Kansas	3,792.7	4,458,185.58	1,680,581.32	66	265.8	122.6	388.4	1,077,611.14	89,013.71	102.0	12.1	114.1	349,958.22	Kansas
Kentucky	1,879.6	4,714,099.38	1,717,805.82	54	211.1	118.2	329.3	134,410.69	40,800.00	2.8		2.8	153,098.85	Kentucky
Louisiana	1,603.7	6,347,853.95	2,831,064.79	57	47.1	28.3	75.4	924,128.64	371,437.65	5.9		5.9	8,644.29	Louisiana
Maine	818.9	2,343,014.91	495,125.23	66	70.2	.3	70.5	113,343.37	41,181.46	.2	1.9	2.1	72,743.65	Maine
Maryland	827.4	1,140,554.27	427,636.[illegible]	88	48.5	3.2	51.7	574,970.90	29,400.00	21.8		21.8	32,360.11	Maryland
Massachusetts	867.1	4,654,918.21	1,187,127.43	60	63.2	4.9	68.1	158,206.29	30,000.00	3.4		3.4	30,738.38	Massachusetts
Michigan	2,289.7	7,208,[illegible]35.82	2,935,734.95	66	290.2	103.0	393.2	1,282,075.91	420,725.00	88.1	1.4	89.5	120,096.19	Michigan
Minnesota	4,318.9	4,645,947.47	133,628.91	84	149.7	139.5	289.2	114,649.27	100.00	1.3	.4	1.7	47,190.24	Minnesota
Mississippi	1,828.0	5,714,491.86	2,814,065.33	61	204.9	74.9	279.8	673,667.22	320,828.04	19.2	4.2	23.4	3,605,579.07	Mississippi
Missouri	3,180.8	2,901,425.33	671,445.44	75	125.8	11.0	136.8	1,337,847.95	281,719.71	68.0		68.0	307,560.13	Missouri
Montana	2,730.3	7,219,244.17	4,048,652.81	55	551.8	244.8	796.6	667,451.08	375,119.26	78.6	11.9	90.5	195,371.64	Montana
Nebraska	4,275.9	5,727,947.49	2,693,488.24	42	162.1	130.7	292.8	62,693.01	23,086.84	8.4		8.4	298,699.33	Nebraska
Nevada	1,343.5	1,635,594.70	745,167.04	85	29.7	118.3	148.0	349,536.98	10,444.96	9.1	9.3	18.4	60,955.22	Nevada
New Hampshire	430.1	911,667.00	391,047.21	71	24.8	3.5	28.3	88,104.15	35,241.66		.6	.6	112,615.99	New Hampshire
New Jersey	629.2	6,215,931.90	2,132,266.20	60	64.2		64.2						65,234.22	New Jersey
New Mexico	2,221.3	2,934,756.32	1,343,845.14	54	168.9	86.3	255.2	274,939.21	129,753.60	10.8	5.2	16.0	267,392.19	New Mexico
New York	3,497.0	17,355,694.40	5,314,545.00	48	455.7	28.3	484.0	2,338,808.60	872,085.74	51.3	3.6	54.9		New York
North Carolina	2,259.3	3,084,111.93	1,538,594.73	41	346.2	23.6	369.8	1,734,589.62	868,977.3	227.9	.3	228.2	1,584,117.25	North Carolina
North Dakota	5,188.8	4,610,632.97	1,942,330.22	52	390.5	444.9	835.4	915,977.20	298,951.00	68.6	253.3	321.9	222,492.47	North Dakota
Ohio	2,996.3	7,898,433.27	2,240,055.09	77	198.7	69.6	268.3	2,409,334.00	461,688.63	32.7	6.4	39.1	233,614.00	Ohio
Oklahoma	2,405.7	3,940,027.91	1,931,330.12	79	187.9	48.5	236.4	2,839,958.50	583,587.00	159.3	38.3	197.6	434,169.17	Oklahoma
Oregon	1,605.5	3,859,713.97	1,598,090.97	55	115.6	81.3	196.9	532,222.22	194,254.87	13.4	15.8	29.2	109,262.95	Oregon
Pennsylvania	3,136.1	11,790,658.36	3,617,335.39	63	431.1	14.1	445.2	1,753,304.53	571,988.81	58.4		58.4	26,339.82	Pennsylvania
Rhode Island	269.8	959,110.79	298,191.64	27	21.8	4.3	26.1	199,751.94	74,618.00	2.8	.2	3.0	24,931.99	Rhode Island
South Carolina	1,353.4	3,513,090.82	1,326,892.73	54	162.5	157.2	319.7	77,398.09	10,787.67		3.4	3.4		South Carolina
South Dakota	4,137.9	4,116,695.40	1,820,707.28	76	275.6	252.3	527.9	357,389.43	59,470.74	65.4		65.4	55,377.27	South Dakota
Tennessee	1,678.7	5,019,033.23	2,508,498.53	31	161.0	61.5	222.5	700,835.12	350,417.54	22.6	5.7	28.3	549,329.74	Tennessee
Texas	7,800.9	17,322,985.27	6,984,132.52	62	738.8	439.2	1,178.0	5,244,063.51	1,459,443.93	247.9	103.8	351.7		Texas
Utah	1,248.9	1,646,827.64	695,811.63	70	94.0	79.4	173.4	389,214.72	116,975.97	24.8	15.6	40.4	211,909.93	Utah
Vermont	384.6	524,908.90	61,754.29	63	27.8		27.6						16,037.73	Vermont
Virginia	1,944.0	4,173,434.50	1,850,900.30	57	202.6	41.3	243.9	1,161,242.97	493,230.78	41.2	32.5	73.7	310,231.84	Virginia
Washington	1,326.4	2,433,262.83	798,612.10	42	104.0	1.1	105.1	946,662.00	160,000.00	12.0	4.5	16.5	94,532.16	Washington
West Virginia	913.1	3,577,911.03	1,336,621.05	64	125.1	8.4	133.5	32,291.00	16,145.50				194,006.53	West Virginia
Wisconsin	2,738.6	6,567,626.26	1,380,806.56	78	173.6	96.1	269.7	52,061.50	21,500.00		.9	.9	63,391.59	Wisconsin
Wyoming	2,037.4	3,275,883.26	1,220,583.53	80	268.3	186.8	455.1	29,783.45	1,241.51	5.0	4.4	9.4	4,053.27	Wyoming
Hawaii	78.4	2,176,281.06	1,179,014.06	45	60.1		60.1	290,788.83	219,999.72	5.9		5.9	749,436.92	Hawaii
TOTALS	105,055.3	252,371,911.47	95,883,774.88	62	9,347.3	3,953.4	13,300.7	40,180,451.88	11,903,160.78	1,880.8	608.6	2,489.4	18,074,944.62	TOTALS
Construction completed		141,875,000	59,555,000											
Balance uncompleted		110,497,000	36,329,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal-aid in the initial improvement. In general, such additional work consists of the construction of a surface of higher type than was provided

U S GOVERNMENT PRINTING OFFICE 19[illegible]

AGRICULTURAL ECONOMICS LIBRARY U.S. DEPARTMENT OF AGRICULTURE

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 2 APRIL 1933

A GRAVEL ROAD AT DEER LAKE, WISCONSIN

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 2 April 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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Because of the necessarily limited edition of this publication it is impossible to distribute it free to any persons or institutions other than State and county officials actually engaged in planning or constructing public highways, instructors in highway engineering, and periodicals upon an exchange basis. At the present time additions to the free mailing list can be made only as vacancies occur. Those desiring to obtain PUBLIC ROADS can do so by sending $1 per year (foreign subscription $1.50) or 10 cents per single copy, to the Superintendent of Documents, United States Government Printing Office, Washington, D.C.

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

THE WISCONSIN FINANCIAL SURVEY

Digest of a Survey of the Finances of Wisconsin in 1930, with Special Reference to Highways, Conducted by the Bureau of Public Roads and the University of Wisconsin

THE BUREAU of Public Roads in cooperation with the University of Wisconsin has recently been conducting a series of studies in a number of States involving the expenditures for highway purposes and the incidence of taxes for these and other purposes. During the late months of 1930 and all of 1931 such an investigation was made in Wisconsin.[1]

This summary of the voluminous report which resulted has been prepared by the Division of Highway Transport of the Bureau in order that the results may be made available insofar as they apply to the general problem of highway taxes, the distribution of expenditures, and the relation of both to the use of highways. Other tax relationships are incidentally revealed.

The calendar year 1930 was chosen for study because it was a census year and the latest year for which data were available. In some cases, the system of public records required that the fiscal year ending in 1930 be used. The variations in comparisons due to using other than the calendar year for these items are known to be negligible, and probably less than they would have been had arbitrary adjustments to the calendar year basis been made.

To classify the incidence of taxation, the State was divided into groups of governmental units and into classes of residence. To distribute expenditure, the highway classifications in use in the State were adopted.

With respect to classes of residence, the State was divided into towns (comprising all rural areas) and incorporated places, the latter being classified as follows:

Places having a population up to 2,500.
Places having a population of 2,500 to 15,000.
Places having a population of 15,000 to 75,000.
Places having a population of 75,000 to 300,000.[2]
Places having a population of 300,000 upward.[3]

Certain of the smaller incorporated places are known as villages, but all are included in the classification given above. In the portion of the report dealing with motor vehicles and motor-vehicle travel the village classification was used.

THE TAXING SYSTEM DESCRIBED

All taxation in the State is under the general supervision of the State tax commission, a paid commission consisting of three members appointed by the Governor. The State income tax and the inheritance tax are administered entirely by the commission. The valuation tax on railroads, car companies, etc., and on all public service corporations operating in more than one governmental district, is also determined by the commission. Most of these taxes are retained by the State.

The valuations used throughout the study are the valuations of the State tax commission and all tax rates are applied to these valuations. It is conceded that the valuations are not quite 100 percent of true 1930 property values, but they are probably closer to it than in most States, on the average, and are well proportioned as between the several counties.

The State property taxes are allotted to the counties on the State tax commission's basis of valuations. The State and county taxes are allocated to the various local governmental units on the valuations as equalized by the county boards. The local units finally collect the State, county, and local property taxes on the basis of the local valuation. In order to put the facts relating to taxation on a parity for the purposes of this study, all property taxes have been computed on the basis of the State valuation.

Throughout the investigation, effort was made to eliminate duplicate and agency transactions from the receipts and expenditures of each class of units of government. For bookkeeping and accounting purposes, each treasurer must enter all moneys received as a receipt, and all moneys paid out as an expenditure. In many cases, involving very large amounts, receipts pass through the hands of several treasurers before they are finally paid out to meet an actual public expenditure and to that extent increase unduly the book receipts and expenditures of public funds.

The entire study was governed by the following rules:

1. To consider as taxes of each class of units of government only those taxes levied by act of its proper officials.

2. To consider as receipts only those funds finally credited as having been expended (not transferred) by officials of that unit of government.

3. To consider as expenditures only those funds disbursed (not transferred) under the direction of the officials of that unit of government in or for that unit of government.

Four governmental purposes only were recognized in the study, and all items were allocated under one or the other of these. These purposes were as follows:

1. *Highways.*—All items having to do with the construction, maintenance, marking and signing, and administration of all public highways.

2. *Education.*—All items having to do with the construction, maintenance, teaching, and administration of all public schools, and also of libraries.

3. *Public protection and benefit.*—All items having to do with the protection of lives and property and the pleasure of the people, including police and fire protection, courts and their officials, parks and playgrounds, and charitable and penal activities.

4. *Government.*—All items having to do with the general administration of those public affairs of a unit of government which are not capable of allocation to one of the three public purposes mentioned above.

POPULATION AND ASSESSED VALUATION

In 1930 Wisconsin had 71 counties, subdivided into 1,280 towns (rural units of government), 359 incorporated villages, and 144 incorporated cities. Milwaukee

[1] The investigation was under the immediate direction of Henry R. Trumbower, professor of economics at the University of Wisconsin; tabulations and analyses of highway and financial data were prepared by the late A. R. Hirst, engineer, and H. R. Briggs, statistician.

[2] No incorporated places of this classification appear in Wisconsin.

[3] The city of Milwaukee is alone in this class in Wisconsin.

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city and County was the only metropolitan area, although there were 18 other cities having a population of over 15,000. Of the population of 2,939,006 in 1930, 37.1 percent lived in the towns (rural areas); 10 percent in incorporated places having a population of 2,500 or less; 12.4 percent in places having a population of 2,500 to 15,000; 20.8 percent in places having a population of 15,000 to 75,000; and 19.7 percent in Milwaukee city, which had a population of 578,249. In short, 37.1 percent of the population was rural and 62.9 percent was urban.

The assessed valuation of all taxable property in Wisconsin in 1930 was approximately $5,896,513,000. Thirty-seven percent of the assessed valuation was in the towns; 7.3 percent in incorporated places having a population of 2,500 or less; 10.1 percent in places having a population of 2,500 to 15,000; 21.9 percent in places having a population of 15,000 to 75,000; and 23.7 percent was in Milwaukee city. Thus, 37 percent of the valuation was in the rural areas, and 63 percent in the urban areas. Of the total valuation, 61.1 percent was in the 13 highest valued counties, which comprise only 13.2 percent of the total area of the State.

LARGE AREAS OF LAND YIELD NO TAXES

An effort was made to ascertain the area of lands upon which taxes were not paid in 1930, but without results definite enough to publish. However, the consensus of opinion of those best informed is that taxes were not paid in 1930 on 4,000,000 to 5,000,000 acres of land previously privately held. This would mean probably 7,000 square miles, or about one eighth of the whole area of Wisconsin. This acreage lies almost entirely in the northern part of the State and in the central sandy area. In addition, the Federal Government and the State now own about 500,000 acres, or 800 square miles, in parks, forest areas, and Indian reservations lying almost entirely in the same areas.

HIGHWAY SYSTEMS IN 1930

In 1930 there were three highway systems in the State: (1) The State trunk highway system consisting of 10,218 miles, which was again divided into 2,345 miles of primary Federal-aid highways, 3,246 miles of secondary Federal-aid highways, and 4,627 miles of other State trunk highways; (2) the county trunk highway system consisting of 13,827 miles; and (3) the remaining roads and streets, for the most part strictly local roads, amounting to 65,494 miles. Of this mileage, 58,934 miles were local town roads, 1,567 miles were village streets, and 4,993 miles were city streets. A summary and percentage relationship is given in table 1.

TABLE 1.—*Classification of highways, roads, and streets in 1930*

Type	Miles	Percentage of total
State highway system	10,218	11.4
County trunk highway system	13,827	15.4
Town roads	58,934	65.8
Village streets	1,567	1.8
City streets	4,993	5.6
Total	89,539	100.0

There were 39,643 miles of surfaced rural highways in Wisconsin on December 30, 1930. This was 47.8 percent of all rural highways. Of this mileage, 3,576 miles (9 percent) were concrete or its equivalent; 781 miles (2 percent) were bituminous macadam; 35,286 miles (89 percent) were crushed stone, gravel, and other minor surfacings. Following are the mileages surfaced and percentage of all mileage surfaced on the several rural highway systems:

	Miles surfaced	Percentage surfaced
Federal-aid highways (primary)	2,341	99.8
Federal-aid highways (secondary)	3,090	95.2
Other State trunk highways	3,932	85.0
Whole State trunk highway system	9,363	91.6
County trunk highways	9,283	67.1
Local town roads	20,997	35.6

Out of the 4,357 miles of high-type surfaces, 3,714 miles (85.2 percent) were on the State trunk highway system; 630 miles (14.5 percent) were on the county trunk highway system; and 13 miles (0.3 percent) were on local roads.

One county (Milwaukee) had every mile of road in the county surfaced, with 30 percent surfaced with concrete or its equivalent. The least populous group of counties had only 26 percent of its mileage surfaced. The most populous group after Milwaukee County had 80 percent of its mileage surfaced, of which about one sixth was of high type.

PERSONAL REPORTS USED TO OBTAIN DATA ON MILEAGE TRAVELED AND INCIDENCE OF MOTOR-VEHICLE TAXATION

As it was the general purpose of the study to determine the amount of the various taxes and imposts, who paid them, where the money was expended and for what purpose, and to disclose the relationships between the various taxes and expenditures, it was necessary to ascertain the distribution of automotive revenues and gasoline taxes paid. For this purpose, personal reports were gathered from 5,116 vehicle owners to determine the mileage traveled on the different classifications of highways. A distribution of revenue was then made on this basis.

Only those owners were questioned whose motor vehicles operated through the whole of 1930. The investigators were instructed to interview owners who would readily respond, selected from all parts of the unit of government being surveyed and from all walks of life. They were instructed to get in the rural districts a proper proportion of motor vehicles as between farmers living on each of the three road systems. Especial and painstaking efforts were to be made to determine accurately, as far as possible, the total mileage traveled by each motor vehicle and its gasoline consumption in Wisconsin in 1930. These were to be carefully estimated or established from all the known facts.

RECEIPTS FROM TAXATION ANALYZED

The total receipts from all taxes and imposts in 1930 were $211,679,900, of which $126,998,400 were derived from the general property tax, $12,976,300 from motor vehicle and drivers' license fees, $7,947,200 from the gasoline tax, and $63,758,000 from other miscellaneous sources.

THE GENERAL PROPERTY TAX

The total general property tax levied and collected in the tax year 1930 was distributed as shown in table 2.

As all persons in Wisconsin live either in towns (townships) or in incorporated places, the above-mentioned table may be restated by allocating the direct levies by

TABLE 2.—*General property taxes imposed by the various governing units*

Imposed by—	Amount	Percentage of total
State	$4,296,800	3.38
Counties	33,294,800	26.22
Towns	20,663,700	16.27
Places up to 2,500	7,415,100	5.84
Places 2,500 to 15,000	10,941,300	8.62
Places 15,000 to 75,000	21,807,300	17.17
Milwaukee	28,579,400	22.50
Total	126,998,400	100.00

TABLE 3.—*Incidence of total general property tax*

Paid by taxpayers in—	Amount paid	Percentage of total	Tax rate per $100
Towns	$38,148,800	30.04	$1.67
Places up to 2,500	10,789,700	8.50	2.50
Places 2,500 to 15,000	15,123,800	11.91	2.57
Places 15,000 to 75,000	28,980,000	22.82	2.24
Milwaukee	33,956,100	26.73	2.46
Total	126,998,400	100.00	2.13

counties and the State to the municipalities in which the taxes were paid, thus showing the incidence of property taxes on taxpayers in rural areas (towns) and in incorporated places, together with the tax rates. The resulting distribution is shown in table 3.

The actual average total tax rates to produce the funds actually raised by levies upon property are also shown. If all State, county, and local taxation and imposts had been levied as a direct tax upon the true valuation of all taxable property, the approximate average tax rates would have been as follows:

	Per $100 of valuation
Towns	$2.72
Places up to 2,500	4.12
Places 2,500 to 15,000	4.23
Places 15,000 to 75,000	3.91
Milwaukee	4.08
Average for the State	3.54

Property taxes were, therefore, the following percentages of all taxes and imposts paid by residents of these groups of local governmental units:

	Percent
Towns	61.4
Places up to 2,500	60.7
Places 2,500 to 15,000	60.8
Places 15,000 to 75,000	57.3
Milwaukee	60.3
Average for the State	60.2

MOTOR-VEHICLE REGISTRATION AND IMPOSTS

The motor-vehicle license fees and the gasoline tax together provide all funds needed to pay the cost of all State participation in highway work, including State aid and the mileage allowances to all other units of government. The State-retained share of the proceeds of these imposts, together with the Federal aid, is spent in great part upon the State trunk highway system.

There were 794,404 (net) motor vehicles of all types registered in 1930. This was one for every 3.7 persons. There was one automobile for every 4.3 persons, and one motor truck for every 26.7 persons. There were only 601,555 motors vehicles assessed as of May 1, 1930. Probably only about 80 percent of the vehicles registered were located by the assessors. The average assessed value of all motor vehicles was $261. The assessment varied from an average of $189 in the towns to $360 in Milwaukee. The property tax on motor vehicles was abandoned in 1931, when the 4-cent gasoline tax became effective. The per capita ownership of motor vehicles was highest in the towns where it was one to every 3.3 persons. In Milwaukee it was one to every 4.2 persons.

The total motor-vehicle license fees paid in 1930 were $12,976,308. The total cost of licensing (including title certificates and drivers' licenses) was $851,244, which averaged $1.07 per motor vehicle registered. About $26,000, collected as drivers' licenses, etc., is included in the above. The cost of collection was 6.56 percent of all the fees collected. The average fee paid for passenger automobiles was $13.08; for motor trucks, $26.50; for city busses, $275; for intercity busses, $386; for publicly owned vehicles, $1. The average fee paid for motor vehicles privately owned and registered was about $16.50. The average for all vehicles, including those publicly owned, was $16.33. In general, the average fee paid increased as the place of residence became more urban. Residents of the towns paid the lowest average fee on all classes of vehicle.

The total (net after refunds) gasoline tax paid (at the 1930 rate of 2 cents per gallon) was $7,947,193. The cost of collecting this tax and making the refunds was $16,301, or about 0.2 percent of the net receipts. The average gasoline tax paid in 1930 for each motor vehicle licensed was $10, indicating an average taxed consumption of 500 gallons per motor vehicle.

The average motor vehicle license fee plus the average gasoline tax paid was $26.33 per motor vehicle.

Of the total amount paid for the registration of motor-vehicles, 36.28 percent was paid by residents of the towns; 41.69 percent by residents of villages and of cities other than Milwaukee; and 22.03 percent by residents of Milwaukee.

While the percentage of the gasoline tax paid by motor-vehicle owners in each class of governmental unit is less precisely known than the percentage of the license fees paid, it can be stated with considerable confidence that, of the total gasoline tax paid by Wisconsin owners, approximately 30 percent is paid by residents of the towns; 48 percent by residents of villages and of cities other than Milwaukee; and 22 percent by residents of Milwaukee.

INCIDENCE OF MOTOR-VEHICLE AND GASOLINE TAXES

Table 4 shows the probable distribution of the amounts paid for motor-vehicle registration fees and for motor-fuel taxes by motor-vehicle owners residing in the several classes of units of government. This tabulation is based on a study of the actual place of ownership and the fees paid by over 20,000 of the motor vehicles registered in Wisconsin in 1930. Unfortunately, the State did not in 1930 require as a prerequisite to registration that the applicant for a license name the local unit of government in which he lived. This study, however, carefully located the local place of residence of the above number and ascertained the fee each paid, and it is felt that the percentages of motor vehicles owned in the various local units of government and of the fees paid are very close to the probable facts in Wisconsin.

TABLE 4.—*Distribution of amounts paid for motor vehicle registration fees and for motor fuel taxes by units of government*

Item	Unit of government					
	State total	Towns	Places up to 2,500	Places 2,500 to 15,000	Places 15,000 to 75,000	Milwaukee
Motor vehicles:						
Percentage of total owned	100.00	41.84	10.51	10.31	20.10	17.24
Number	794,404	332,379	83,492	81,903	159,675	136,955
License fees:						
Percentage of total paid	100.00	36.28	9.60	10.93	21.16	22.03
Amount	$12,976,300	$4,707,800	$1,245,700	$1,418,300	$2,745,800	$2,858,700
Average per vehicle	$16.33	$14.16	$14.92	$17.32	$17.20	$20.87
Gasoline consumption:						
Average gallons per vehicle	[1] 563	[1] 409	[1] 618	[1] 675	[1] 690	[1] 734
Total gallons (in millions)	397.36	119.07	45.22	48.45	96.54	88.08
Gasoline tax:						
Percentage of total paid	100.00	29.96	11.38	12.19	24.30	22.17
Amount	$7,947,200	$2,381,400	$904,300	$968,900	$1,930,800	$1,761,700
Average per vehicle	$10.00	$7.16	$10.83	$11.83	$12.10	$12.86
Payments:						
Total fees and gasoline tax	$20,923,500	$7,089,200	$2,150,100	$2,387,200	$4,676,600	$4,620,400
Percentage of total paid	100.00	33.88	10.27	11.41	22.35	22.09
Average per vehicle	$26.33	$21.32	$25.75	$29.15	$29.30	$33.73

[1] This is the average gasoline consumption for motor vehicles in operation the whole of 1930 as determined by the survey. All other figures below these are modified to accord with the actual motor-fuel tax paid.

As to gasoline consumption and gasoline tax paid, these were derived from the survey of traffic. The actual sums paid were computed upon the basis of the actual gasoline tax paid, and not upon the gasoline consumption indicated as a result of the traffic survey. In other words, there was the fixed fact that so many dollars in gasoline tax were paid (less refunds) and subtracting the estimated amount for tourist payments, the remainder was divided in proportion to the relative consumption indicated by the traffic survey.

The data given in table 4 are summarized in table 5, which indicates quite conclusively that both motor-vehicle license fees and gasoline taxes fall in much heavier ratio on the urban communities than on the rural communities. This indicates that the type of motor vehicle owned in the rural communities probably averages lighter in weight than in the urban communities, besides the fact that the average mileage traveled is less than in the urban communities.

TABLE 5.—*Distribution of amounts paid for motor vehicle registration fees and for motor fuel taxes by units of government*

Paid by motor vehicle owners in—	Motor vehicle fees	Gasoline tax	Total	Percent	Per vehicle
Towns	$4,707,800	$2,381,400	$7,089,200	33.88	$21.32
Places up to 2,500	1,245,700	904,300	2,150,100	10.27	25.75
Places 2,500 to 15,000	1,418,300	968,900	2,387,200	11.41	29.15
Places 15,000 to 75,000	2,745,800	1,930,800	4,676,600	22.35	29.30
Milwaukee	2,858,700	1,761,700	4,620,400	22.09	33.73
Total for State	12,976,300	7,947,200	20,923,500	100.00	26.33

It has been customary to estimate the amount of motor-vehicle license fees paid and gasoline tax paid on the sole basis of the number of motor vehicles registered, but apparently the use of this method leads to glaring inaccuracies as to tax payments.

For example, it will be noted that while the average payment per motor vehicle in the towns in Wisconsin in 1930 was $21.32, the average payment by motor vehicles owned in Milwaukee was $33.73. The payments per motor vehicle averaged $12.41 (58 percent) higher in Milwaukee than in the towns. The average payments of gasoline tax were about 80 percent larger in Milwaukee than in the towns.

TAXATION DATA SUMMARIZED

In table 6 the amounts paid by taxpayers in the several classes of governmental unit are summarized. It is shown in this table that of all taxation for all purposes in all units of government, 29.3 percent was paid by taxpayers in the towns; 8.4 percent by taxpayers in places having a population of 2,500 or less; 11.8 percent by taxpayers in places having a population of 2,500 to 15,000; 23.9 percent by taxpayers in places having a population of 15,000 to 75,000; and 26.6 percent by taxpayers in Milwaukee. It is thus evident that, of all taxation, 29.3 percent fell upon taxpayers in rural areas, and 70.7 percent upon taxpayers in urban communities.

TABLE 6.—*Incidence and classification of all taxes paid in 1930*

Paid by taxpayers in—	General property tax		Registration fees, drivers' licenses, and gasoline tax		Miscellaneous taxes, fees, and imposts		All taxes	
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Towns	$38,148,800	30.1	$7,089,200	33.9	$16,888,400	26.5	$62,126,400	29.3
Place up to 2,500	10,789,700	8.5	2,150,100	10.3	4,809,800	7.6	17,749,600	8.4
Places 2,500 to 15,000	15,123,800	11.9	2,387,200	11.4	7,385,100	11.6	24,896,100	11.8
Places 15,000 to 75,000	28,980,000	22.8	4,676,600	22.3	16,861,400	26.4	50,518,000	23.9
Milwaukee	33,956,100	26.7	4,620,400	22.1	17,813,300	27.9	56,389,800	26.6
		100.0		100.0		100.0		100.0
Total for State	126,998,400	60.0	20,923,500	9.9	63,758,000	30.1	211,679,900	100.0

Of the total revenue obtained by taxation for all purposes, $211,679,900, it is found that 60.0 percent was contributed by the general property tax, 9.9 percent by the general taxation on motor vehicles and motor fuel, and 30.1 percent by miscellaneous taxes, fees, and imposts.

The funds described in table 6 as "Miscellaneous taxes, fees, and imposts" are subdivided as follows:

	Amount	Percentage of total State taxes
State-imposed taxes, fees, and imposts:		
Ad valorem tax on public utilities:		
Statewide	$6,908,700	6.8
Local	7,392,500	
Special imposts on motor vehicles:		
Bus and truck permits	47,100	.2
Ton-mile tax	91,000	
Gasoline inspection	236,200	
Income tax	21,454,400	10.1
Inheritance tax	2,645,700	13.0
Taxes on insurance companies:		
Life	1,555,600	
Other	845,300	
Miscellaneous taxes:		
General	504,300	
Local	913,100	
Miscellaneous fees, licenses, and permits	2,124,200	
Miscellaneous collections by local units	19,039,900	
Total	63,758,000	30.1

If the percentages given in this tabulation are combined with those of table 6, it will be found that of all taxation in all units of government (including all fees, licenses, and imposts) 66.8 percent was obtained by taxation on real and personal property (including the personal property tax on motor vehicles, $3,300,000) and all State ad valorem taxes; 10.1 percent by licenses, fees, and taxes on motor vehicles and their use; 10.1 percent by taxation on incomes; and 13 percent by miscellaneous fees, licenses, permits, and imposts.

Of all taxation and imposts, 33 percent was imposed by act of State officials; 20.7 percent by county officials; 9.9 percent by the people or officials of towns; 3.8 percent by the people or officials of places having a population of less than 2,500; 5.7 percent by officials of places having a population of 2,500 to 15,000; 11.8 percent by officials of places having a population of 15,000 to 75,000; and 15.1 percent by officials of Milwaukee.

HIGHWAY REVENUES OBTAINED FROM GENERAL PROPERTY AND MOTOR-VEHICLE TAXES

Of the total of $126,998,400 obtained from the general taxation of property in 1930, the sum of $31,888,200 was imposed specifically for highway purposes. In this sum are included special assessments on property to the amount of $4,700,700. The income from the taxation of motor vehicles, including the vehicle and drivers' license fees, gasoline tax, and special fees and permits, is devoted entirely to highway purposes. The total revenue from these sources, $21,297,800, provided, as stated previously, all funds for State participation in highway work, including State aid and the mileage allowances to all other units of government.

In considering highway taxation and expenditures, it was necessary to fix and select certain definite charges uniformly in all governmental units in order to make a true comparison, and throughout the report highway items include all charges for construction, maintenance, marking and signing, and the administrative costs of the various bodies in authority properly allotable to roads. On this basis, the total highway taxes were as shown in table 7.

These taxes were levied on the various units in the amounts shown in table 8.

BONDED INDEBTEDNESS

The State has a constitutional prohibition against bonding and has no bonded indebtedness for any purpose except $1,363,700 of Civil War bonds still unpaid. All other units of government may bond.

The total of all publicly issued bonds outstanding on December 31, 1930, as nearly as could be determined at the time, was $154,378,400. The total debt service on these bonds in 1930 was $43,041,300. Table 9 gives the distribution of this indebtedness according to the governmental purposes for which the bonds were issued. The table also lists the indebtedness of each class of government unit.

The total bonds outstanding on December 31, 1930, were 2.6 percent of the total assessed valuation of the State and the per capita indebtedness was $52.53. The per capita cost of all debt service in 1930 was $14.64.

The bonds outstanding issued for highway purposes amounted, at the close of 1930, to $59,879,200. The county highway bonds outstanding totaled $43,017,500. The remaining $16,861,700 of highway bonds were issued by cities and villages for street construction and

WISCONSIN TAXES IN 1930

The total receipts from all taxes and imposts in Wisconsin in 1930 were $211,679,900. Receipts from the taxation of general property were $126,998,400; from motor-vehicle fees and gasoline tax, $20,923,500; from special imposts on motor vehicles, $374,300; from other sources, including income and inheritance taxes, valuation tax on railroads and other public utilities, and miscellaneous fees and imposts, $63,383,700.

Of the total revenues from all taxes and imposts, taxpayers in the towns contributed $62,126,400 or 29.3 percent; those in incorporated places having a population of 2,500 or less, $17,749,600, or 8.4 percent; in places of 2,500 to 15,000 population, $24,896,100, or 11.8 percent; in places of 15,000 to 75,000 population, $50,518,000, or 23.9 percent; in Milwaukee, $56,389,800, or 26.6 percent.

The average actual tax rates on general property, per $100 valuation, were as follows: Towns, $1.67; places of 2,500 population or less, $2.50; places 2,500 to 15,000, $2.57; places 15,000 to 75,000, $2.24; Milwaukee, $2.46.

Motor-vehicle owners in the towns paid in license fees and gasoline tax an average of $21.32; in places of 2,500 population or less, the average was $25.75; in places 2,500 to 15,000, $29.15; in places 15,000 to 75,000, $29.30; in Milwaukee, $33.73.

TABLE 8.—*All taxes specifically imposed for highway purposes in 1930 classified according to places of collection*

Paid by taxpayers in—	Amount	Percent
Towns	$23,035,200	43.3
Places up to 2,500	4,806,800	9.0
Places 2,500 to 15,000	5,660,700	10.7
Places 15,000 to 75,000	10,650,900	20.0
Milwaukee	9,032,400	17.0
Total	53,186,000	100.0

TABLE 7.—*All taxation specifically imposed for highway purposes in 1930*

	Amount	Percent
TAXES IMPOSED ON GENERAL PROPERTY		
County	$14,557,800	27.4
Local	12,629,700	23.8
Special assessments	4,700,700	8.8
Total	31,888,200	60.0
IMPOSTS UPON MOTOR VEHICLES AND THEIR OWNERS AND OPERATORS		
Vehicle and drivers' license fees	$12,976,300	24.4
Bus and truck permits	47,100	.1
Ton-mile tax	91,000	.2
Motor-fuel tax	7,947,200	14.9
Gasoline inspection	236,200	.4
Total	21,297,800	40.0
Grand total	53,186,000	100.0

TABLE 9.—*Bonded indebtedness as of Dec. 31, 1930, classified by purpose of issue and by unit of government*

BY PURPOSE

	Amount	Percentage of total	Debt service in 1930 (principal and interest)
Highways	$59,879,200	38.8	$12,486,500
Education	29,208,600	18.9	12,300,100
Public benefit	47,961,100	31.1	10,582,500
Government	17,329,500	11.2	7,672,200
Total	154,378,400	100.0	43,041,300

BY UNIT OF GOVERNMENT

Towns	$1,735,300	1.1
Places up to 2,500	6,984,000	4.5
Places 2,500 to 15,000	12,421,000	8.1
Places 15,000 to 75,000	37,090,800	24.0
Milwaukee	45,030,800	29.2
Counties	49,752,800	32.2
State	1,363,700	.9
Total	154,378,400	100.0

improvement, and, in a few instances, by towns. The distribution, as nearly as could be determined, was as follows:

Villages and towns	$1,161,400
Cities other than Milwaukee	9,568,800
Milwaukee	6,131,500

The debt service in 1930 for all outstanding bonds was $43,041,300 of which $12,486,500 was for highway bonds. The debt service for the county highway bonds was $5,027,500, the principal payments being $3,028,100 and the interest $1,999,400. There is no information on the division between principal payments and interest for the debt service on other than county highway bonds. However, the debt service on the remaining $16,861,700 of highway bonds was $7,459,000; and we may obtain an approximate figure by assuming that the average rate of interest was the same as that of the county bonds (4.65 percent). On this basis we obtain a figure of $784,100 for interest and $6,674,900 for principal payments. We may therefore conclude that the debt service on all street and highway bonds in 1930 consisted approximately of $9,703,000 in principal payments and $2,783,500 in interest. If we apply the same rate to the total State indebtedness we obtain the following approximate figures for total debt service: Principal payments, $35,862,700; interest, $7,178,600.

During 1930 $3,028,100 of highway bonds were retired and $4,816,500 in new bonds were issued by the various counties; so that the indebtedness of the counties in highway bonds was increased by $1,788,400 during that year. The extent to which other indebtedness was increased is not recorded in the report.

EXPENDITURES ANALYZED

The total expenditures by all units of government, for all purposes except the payment of principal on bonds, were $202,377,000. Interest payments of approximately $7,200,000 are included in this total. Addition of the principal payments of approximately $35,860,000 would bring the total expenditures to $238,237,000. The principal payments are not to be regarded as true expenditures, since they represent the repayment of funds which were recorded as expenditures in previous years.

Table 10 gives the expenditures by the State, the counties, and the various classes of municipal units, for each of the several purposes of government. Of all expenditures, 30.3 percent were made for highways, 32.6 percent for education, 30.9 percent for public benefit, and 6.2 percent for other governmental purposes.

The distribution of total expenditures according to the classes of governmental unit in which they were made was as follows:

Expended in—	Amount	Percent
Towns	$79,199,900	39.1
Places up to 2,500	14,985,700	7.4
Places 2,500 to 15,000	20,783,700	10.3
Places 15,000 to 75,000	43,141,300	21.3
Milwaukee	44,266,400	21.9
Total	202,377,000	100.0

It is apparent from this tabulation that 39.1 percent of the expenditures were made in the rural areas, and 60.9 percent in the urban areas. Table 11 gives the per capita expenditures made for the several purposes of government in the several classes of local unit of government.

TABLES SHOW RELATIONS BETWEEN VALUATION, TAXATION, AND EXPENDITURES

The valuation, taxation, and expenditure per capita in 1930 were as shown in table 12. The ratio of per capita expenditures to per capita taxation is also given. This ratio is significant as an index of the flow of State (including Federal) and county aids to the less populous areas. For instance, about 21.5 percent of the taxes paid by citizens of Milwaukee were not expended in Milwaukee, while about 27.5 percent of the expenditure in the towns was not provided by taxes paid by inhabitants of these rural areas.

TABLE 10.—*Distribution of total expenditures in 1930, showing the amounts expended by the State, the counties, and the several classes of governmental unit, on highways, education, public benefit, and other governmental purposes*

Unit of government making expenditure	Highways		Education		Public benefit		Government		All purposes		
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Percentage of total
Towns	$10,387,300	34.7	$16,809,000	56.2	$945,300	3.2	$1,776,800	5.9	$29,918,400	100.0	14.8
Places up to 2,500	1,824,000	19.5	4,615,200	49.4	1,980,200	21.2	931,200	9.9	9,350,600	100.0	4.6
Places 2,500 to 15,000	2,089,700	13.7	6,969,300	45.9	4,969,300	32.7	1,172,000	7.7	15,200,400	100.0	7.5
Places 15,000 to 75,000	4,680,000	13.8	16,381,900	48.2	11,074,200	32.6	1,823,100	5.4	33,959,300	100.0	16.8
Milwaukee	4,311,900	12.3	12,401,600	35.5	17,017,700	48.6	1,269,900	3.6	35,001,000	100.0	17.3
Counties	20,310,800	47.2	1,209,000	2.8	17,625,700	41.0	3,915,200	9.0	43,060,600	100.0	21.3
State	17,676,700	49.3	7,541,600	21.0	9,045,600	25.2	1,622,800	4.5	35,886,700	100.0	17.7
Total	61,280,400	30.3	65,927,500	32.6	62,658,100	30.9	12,511,000	6.2	202,377,000	100.0	100.0

TABLE 11.—*Per capita expenditures by purpose and by classes of local unit in which they were made, based on a population of 2,939,006*

Unit of government	Highways	Education	Public benefit	Government	Total
Towns	$41.70	$18.03	$9.47	$3.43	$72.63
Places up to 2,500	12.02	18.61	15.16	5.01	50.80
Places 2,500 to 15,000	7.71	21.14	22.99	5.19	57.02
Places 15,000 to 75,000	8.45	29.79	27.42	4.92	70.58
Milwaukee	7.46	25.21	39.51	4.37	76.55
State	20.85	22.43	21.32	4.25	68.85

TABLE 12.—*Valuation, taxation and expenditures per capita in 1930, based on a population of 2,939,006*

Item	State	Towns	Places up to 2,500	Places 2,500 to 15,000	Places 15,000 to 75,000	Milwaukee
Valuation	$2,006.00	$2,001.00	$1,465.00	$1,625.00	$2,115.00	$2,417.00
Taxation	72.02	56.99	60.17	68.32	82.65	97.52
Expenditures	68.85	72.63	50.80	57.02	70.58	76.55
Ratio of per capita expenditure to per capita taxation	0.956	1.275	0.844	0.835	0.854	0.785

TABLE 13.—*Distribution of population, motor-vehicle ownership, property valuation, taxes paid, and expenditures made in the several groups of local units of government in 1930*

Unit of government	Population	Motor-vehicle ownership	Valuation	Taxes paid	Expenditures made
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Towns	37.1	41.4	37.0	29.3	39.1
Places up to 2,500	10.0	10.6	7.3	8.4	7.4
Places 2,500 to 15,000	12.4	10.3	10.1	11.8	10.3
Places 15,000 to 75,000	20.8	20.2	21.9	23.9	21.3
Milwaukee	19.7	17.5	23.7	26.6	21.9
State	100.0	100.0	100.0	100.0	100.0

Table 13 gives the percentage distribution of population, motor-vehicle ownership, property valuation, taxes paid, and expenditures made in the several groups of local governmental unit. While the trends in all these items are similar, there are significant differences. Thus the towns, with 37.1 percent of the population, paid only 29.3 percent of the taxes, while Milwaukee, with less than 20 percent of the population, paid nearly 27 percent of the taxes. Three factors, a higher per capita valuation, more intensive use of motor vehicles, and higher public benefit costs due to the density of population serve to account for these differences.

It is noticeable that expenditures in the various classes of local unit are rather closely proportional to their valuations. The greatest differences are in the towns where the percentage of expenditure exceeds that of valuation by about 2 percent, and in Milwaukee, where the reverse is true.

The figures for motor-vehicle ownership show that the number of persons owning cars, per 100 of population, is greater in the rural areas than in the cities, particularly Milwaukee.

WISCONSIN EXPENDITURES IN 1930

The total expenditures (exclusive of principal payments on bonds and loans) by all units of government in Wisconsin in 1930 were $202,377,000, of which $61,280,400 were expended for highways and streets, $65,927,500 for education, $62,658,100 for public benefit, and $12,511,000 for other governmental purposes.

Expenditures by the State were $35,886,700; by the counties $43,060,600; by Milwaukee, $35,001,000; by other incorporated places $58,510,300; by the towns, $29,918,400.

Thirty-nine percent of the expenditures were made in rural areas (towns); 7 percent in incorporated places having a population of 2,500 or less; 10 percent in places of 2,500 to 15,000 population; 21 percent in places of 15,000 to 75,000 population; and 22 percent in Milwaukee.

HIGHWAY EXPENDITURES DISCUSSED IN DETAIL

The total expenditures for highways and streets in Wisconsin in 1930 were $61,280,400. This amount is lower than figures sometimes quoted because special efforts have been made to eliminate all agency transactions and duplicate expenditures. The total shown does not include principal payments on highway bonds (approximately $9,700,000), because in previous years these have already been shown as highway expenditures, and to show them again would double this item of apparent expenditures over a period of years.

The total also does not include purchase of new equipment, gravel pits, etc., by the counties. All of these items are paid for by the counties, but are later charged back to each individual piece of work as a part of the construction and maintenance cost, so that at the end of a series of years this expenditure appears in the construction and maintenance expenditures of the county. In short, each county sets up a revolving fund for machinery, etc., and charges back to work a sufficient fund to recover its cost within its life.

The total highway expenditures do include interest on all bonds and indebtedness (approximately $2,800,000), and all supervision costs and general overheads.

There was considerable transfer of funds as a result of grants of aid. In 1930, the towns and villages received from the State $25 per mile, and the cities from $50 to $200 per mile, according to their population, the largest amount per mile being paid to Milwaukee city. This aid totaled $1,954,000 to all units of government.

The sources of the funds from which the expenditures were made were as follows:

	Amount	Percent
Federal aid	$3,019,700	5.0
Taxation of property	21,694,600	35.4
Imposts on motor vehicles	20,025,800	32.7
Miscellaneous sources	7,048,100	11.5
Proceeds of bond issues and loans	9,462,200	15.4
Total	61,280,400	100.0

Table 14 gives the total expenditures on each of the highway systems of the State and the village and city roads and streets; and the expenditures for construction and maintenance made under State and county supervision on each of these classes of road. The construction and maintenance figures do not include interest paid on bonds or the general overhead cost not charged to the work at the time it was done. These costs are included in the balance shown in the last column of the table.

TABLE 14.—*Expenditures on the several highway systems and the local streets in 1930, including construction and maintenance expenditures under State and county supervision*

Highway system	Mileage in system	Total expended	Percentage of total expenditures	Expenditure under State and county supervision		Balance not accounted for in two preceding columns [1]
				Construction	Maintenance	
Federal aid:						
Primary	2,345	$17,121,500	28.0	$7,021,900	$1,244,700	$1,799,900
Secondary	3,246			5,422,700	1,632,300	
State trunk	4,627	7,797,200	12.7	4,948,000	2,113,900	735,300
Total, State system	10,218	24,918,700	40.7	17,392,600	4,990,900	2,535,200
County trunk	13,827	9,449,100	15.4	3,662,300	4,662,300	1,124,500
Town local	58,934	13,910,900	22.7	2,807,000	466,600	[2] 10,637,300
Village local	1,567	1,590,000	2.6	91,200	3,100	[2] 12,907,400
City local	4,993	11,411,700	18.6			
All highways	89,539	61,280,400	100.0	23,953,100	10,122,900	27,204,400

[1] Interest and general overhead charges are included in this column.

[2] Includes also expenditures made under local supervision.

Of all expenditures on highways and streets in Wisconsin in 1930, there were expended on the State trunk highway system $24,918,700, or 40.7 percent; on the county trunk highway system, $9,449,100, or 15.4 percent; on the town local rural highways, $13,910,900, or 22.7 percent; on local village streets, $1,590,000, or 2.6 percent; on local city streets, $11,411,700, or 18.6 percent. Expenditures on all rural highways were $48,278,700, or 78.8 percent; and on urban and village streets, $13,001,700 (21.2 percent, or $1,982 per mile). These expenditures include interest paid on bonds and the cost of supervision and overhead.

CONSTRUCTION ABSORBS BULK OF EXPENDITURES ON STATE WORK

The figures on construction and maintenance given in table 14 cover the cost of all construction and maintenance on the Federal aid, State trunk, and county trunk highway systems in 1930. On the local roads and streets the great bulk of the expenditures was made by the local authorities, and the work was done under their supervision. Local expenditures administered by the local units were divided as follows:

Towns:	
Construction	$2,899,600
Maintenance	7,068,400
Interest, overhead, etc	419,300
Total	10,387,300
Cities and villages:	
Construction	5,964,700
Maintenance	4,351,000
Interest, overhead, etc	2,589,900
Total	12,905,600

No street or highway work was done in the city of Milwaukee under State or county supervision. Expenditures on roads and streets in that city were $4,311,900.

Of all expenditures for construction and maintenance on the Federal-aid and State trunk systems, 78 percent was spent for construction and 22 percent for maintenance. For all work under State and county supervision the amount expended for construction was 70 percent, and that for maintenance 30 percent. The county system was the only one on which the expenditures for maintenance exceeded those for construction. However, the expenditure on the town roads was predominantly for maintenance, as is shown in the tabulation of expenditures by local authorities, given above.

LOCATION OF EXPENDITURES SHOWN

The following tabulation shows the distribution of highway expenditures according to the classes of local unit in which the expenditures were made:

	Amount	Percent
In towns	$45,452,900	74.2
In places up to 2,500	3,545,400	5.8
In places 2,500 to 15,000	2,805,200	4.6
In places 15,000 to 75,000	5,165,000	8.4
In Milwaukee	4,311,900	7.0
Total	61,280,400	100.0

As would be expected, the major part of the expenditures (74.2 percent) were made in the towns, i.e., on rural roads.

The figures given above, when compared with those given in table 8, show the following ratios between the expenditures made on highways and streets within the various classes of local unit and the taxes paid in those units:

	Ratio of highway expenditures to highway taxes
Towns	1.97
Places up to 2,500	.74
Places 2,500 to 15,000	.50
Places 15,000 to 75,000	.48
Milwaukee	.48
All cities and villages	.53

There were 18 counties out of the 71 in which the total highway taxes collected were greater than the total highway expenditures made in them. In these counties, the excess of highway taxes over highway expenditures was $5,145,600, of which Milwaukee County contributed $2,884,300, or 56.1 percent. Nearly 25 percent of the aggregate highway taxes collected in these counties was expended in other counties. Milwaukee County contributed 26.5 percent of its highway taxes to work in other counties.

TABLE SHOWS ULTIMATE SOURCES OF HIGHWAY FUNDS DERIVED FROM TAXES AND IMPORTS

Of the total expenditures for highways the sum of $48,768,500 was obtained from taxes and imposts paid to the State, county, and local governments. Funds from other sources consisted of $3,049,700 in the form of Federal aid and $9,462,200 from bonds and loans. The proceeds from bonds and loans were expended on the roads and streets of the State in the following amounts:

TABLE 15.—*Approximate amounts of the 1930 taxes and imposts expended on the current highway program, listed according to highway system, type of tax, and class of local unit in which the tax was paid*

IMPOSTS ON GENERAL PROPERTY

Paid by taxpayers in	Highway system										
	State trunk		County trunk		Town local		City and village local		All highways		
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Percentage of total
Towns	$1,616,400	11.7	$3,114,900	22.6	$9,070,700	65.7			$13,802,000	100.0	63.6
Places up to 2,500	317,800	20.9	612,400	40.3	224,400	14.8	$365,000	24.0	1,519,600	100.0	7.0
Places 2,500 to 75,000	910,200	19.0	1,754,000	36.7	642,800	13.4	1,478,200	30.9	4,785,200	100.0	22.1
Milwaukee	87,100	5.5	167,800	10.6	61,500	3.9	1,271,400	80.1	1,587,800	100.0	7.3
Total	2,931,500	13.5	5,649,100	26.0	9,999,400	46.1	3,114,600	14.4	21,694,600	100.0	100.0
IMPOSTS ON MOTOR VEHICLES											
Towns	$5,012,400	73.8	$1,050,300	15.5	$494,700	7.3	$227,400	3.4	$6,784,800	100.0	33.9
Places up to 2,500	1,519,400	73.8	318,400	15.5	150,000	7.3	68,900	3.4	2,056,700	100.0	10.3
Places 2,500 to 75,000	4,994,600	73.8	1,046,500	15.5	493,000	7.3	226,600	3.4	6,760,700	100.0	33.7
Milwaukee	3,268,100	73.8	684,800	15.5	322,500	7.3	148,200	3.4	4,423,600	100.0	22.1
Total	14,794,500	73.8	3,100,000	15.5	1,460,200	7.3	671,100	3.1	20,025,800	100.0	100.0
MISCELLANEOUS IMPOSTS AND TAXES											
Towns					$2,311,300	100.0			$2,311,300	100.0	32.8
Places up to 2,500							$677,300	100.0	677,300	100.0	9.6
Places 2,500 to 75,000							2,818,600	100.0	2,818,600	100.0	40.0
Milwaukee							1,240,900	100.0	1,240,900	100.0	17.6
Total					2,311,300	32.8	4,736,800	67.2	7,048,100	100.0	100.0
ALL IMPOSTS AND TAXES											
Towns	$6,628,800	28.9	$4,165,200	18.2	$11,876,700	51.9	$227,400	1.0	$22,898,100	100.0	46.9
Places up to 2,500	1,837,200	43.2	930,800	21.9	374,400	8.8	1,111,200	26.1	4,253,600	100.0	8.7
Places 2,500 to 75,000	5,904,800	41.1	2,800,500	19.5	1,135,800	7.9	4,523,400	31.5	14,364,500	100.0	29.5
Milwaukee	3,355,200	46.3	852,600	11.7	384,000	5.3	2,660,500	36.7	7,252,300	100.0	14.9
Total from taxes and imposts	17,726,000	36.4	8,749,100	17.9	13,770,900	28.2	8,522,500	17.5	48,768,500	100.0	100.0
Federal aid	3,049,700								3,049,700		
Total	20,775,700	40.1	8,749,100	16.9	13,770,900	26.6	8,522,500	16.4	51,818,200	100.0	

Federal-aid system	$2,500,000
State trunk system	1,643,000
County trunk system	700,000
Town roads	140,000
Village streets	411,000
City streets	4,068,200
Total	9,462,200

Table 15 gives the results of an analysis the purpose of which was to determine the amounts contributed by taxpayers in the various classes of local unit to the funds expended on the several highway systems and the village and city streets. The Federal-aid money, coming directly from the United States Government, is naturally not subject to analysis on this basis. It is possible to forecast the manner in which borrowed money will be repaid, and thus allocate the proceeds of bond sales to the various local units, but such repayments have nothing to do with taxes paid in 1930. For that reason it was thought best to restrict the analysis to the $48,768,500 which was actually derived from taxes and imposts.

The motor-vehicle imposts were allocated to the various local units according to the percentages given in table 5, which are based on the results of the questionnaire survey and the analysis of registration statistics. The allocation of the general property taxes and miscellaneous imposts was accomplished by determining as closely as possible, from the data obtained in the survey, the ultimate sources of all taxes and imposts raised by the county and local governments and spent on the several road systems and the village and city streets. The results of the analysis are necessarily approximate, but are believed to represent fairly accurately the true situation in 1930.

Many significant relationships can be derived from a study of table 15, particularly when it is analyzed in connection with the data regarding travel on the various highway systems, which is presented and discussed in the following paragraphs.

AVERAGE MILEAGE TRAVELED AND GASOLINE CONSUMPTION

The average mileage traveled and the average gasoline consumption in Wisconsin by Wisconsin motor vehicles in 1930 are given in table 16. The ownership of these motor vehicles is classified according to the place of residence of the owners. The mileage traveled for each of these classes is divided as between roads and streets located in towns (rural sections), in villages, and in incorporated cities. The data are also classified by type of vehicle. These facts were determined by means of the personal reports or schedules obtained from 5,116 motor-vehicle owners, as described on page 22.

About two thirds of all motor-vehicle traffic takes place in the rural districts, and about one third in the urban districts, including villages. According to table 16, 64.1 percent of the average miles traveled occurred in the towns, 5.2 percent in the villages, and 30.7 percent in the cities. The automobiles and trucks owned in towns traveled, on the average, 5,751 miles in 1930;

TABLE 16.—*Average mileage traveled and average gasoline consumption in Wisconsin by Wisconsin motor vehicles in 1930*

Place of ownership and kind of motor vehicle	Reports tabulated	Average annual gasoline consumption		Average annual travel			
				Totals	In towns	In villages	In cities
		Gallons	Miles per gallon	Miles	Miles	Miles	Miles
Towns:							
Automobiles	1,903	376	14.8	5,583	4,787	298	498
Motor trucks	269	635	10.9	6,924	5,836	325	763
Both automobiles and motor trucks	2,172	409	14.1	5,751	4,917	302	532
Percentage	42.5			100.0	85.5	5.2	9.3
Villages:							
Automobiles	474	536	13.9	7,467	5,871	905	691
Motor trucks	90	938	10.8	10,147	7,633	1,714	800
Both automobiles and motor trucks	564	600	13.2	7,895	6,152	1,034	709
Percentage	11.0			100.0	77.9	13.1	9.0
Cities other than Milwaukee:							
Automobiles	1,382	615	14.1	8,671	4,942	358	3,371
Motor trucks	216	1,060	10.5	11,103	5,595	501	5,008
Both automobiles and motor trucks	1,598	675	13.3	8,999	5,030	377	3,592
Percentage	31.2			100.0	55.9	4.2	39.9
Milwaukee:							
Automobiles	689	716	13.1	9,370	3,526	218	5,626
Motor trucks	93	867	10.4	9,027	1,553	145	7,329
Both automobiles and motor trucks	782	734	12.7	9,330	3,292	210	5,828
Percentage	15.3			100.0	35.3	2.2	62.5
Total for State:							
Automobiles	4,448	520	14.1	7,329	4,755	369	2,205
Motor trucks	668	851	10.6	9,002	5,404	544	3,054
Both automobiles and motor trucks	5,116	563	13.4	7,549	4,841	392	2,316
Percentage	100.0			100.0	64.1	5.2	30.7

85.5 percent of this mileage was on rural roads, 5.2 percent on village streets, and 9.3 percent on city streets. The village-owned automobiles and trucks traveled 7,895 miles in 1930, on the average; 77.9 percent of this mileage was on rural roads, 13.1 percent on village streets, and 9 percent on city streets.

The city-owned motor vehicles are divided between the vehicles owned in Milwaukee, and those owned in the other cities of the State. The average 1930 mileage of all the motor vehicles owned in the cities, not including Milwaukee, amounted to 8,999 miles; 55.9 percent of this mileage was on the rural roads, 4.2 percent on village streets, and 39.9 percent on city streets. The average mileage of motor vehicles owned in Milwaukee somewhat exceeds that of vehicles owned in the other cities. The motor vehicles owned in Milwaukee averaged 9,330 miles of travel; 35.3 percent of this was on rural roads, 2.2 percent on village streets, and 62.5 percent on city streets. It may be inferred that a very large part of this travel on city streets occurred in Milwaukee itself. The Milwaukee average of 9,330 miles per year is 62 percent in excess of the average for town-owned vehicles, 5,751 miles. In general, automobiles owned in incorporated cities and villages produced larger average mileages than those owned in rural areas.

The motor-truck mileage for the State as a whole averages 9,002 miles, which is 23 percent above the average number of miles traveled by automobiles. In each of these ownership classifications, the motor truck is used on the average to a greater degree than the automobile, except in Milwaukee. In Milwaukee, the average miles traveled by automobiles are 9,370, and the average truck mileage is 9,027.

The average gasoline consumption, according to this investigation, amounted to 563 gallons per motor vehicle; for automobiles, the average was 520; and for motor trucks it was 851. The average consumption of gasoline for motor trucks exceeded that of automobiles by 64 percent; the average mileage of motor trucks exceeded that of automobiles by only 23 percent. This difference is accounted for by the fact that the automobiles averaged 14.1 miles per gallon of gasoline, and the motor trucks averaged only 10.6 miles.

The average yearly consumption of 563 gallons of gasoline by motor vehicles operating 12 months in 1930 is the weighted average used by those owned in towns, 409 gallons; in villages, 600 gallons; in cities other than Milwaukee, 675 gallons; and in Milwaukee, 734 gallons. The average mileage per gallon for all motor vehicles was 13.4 miles.

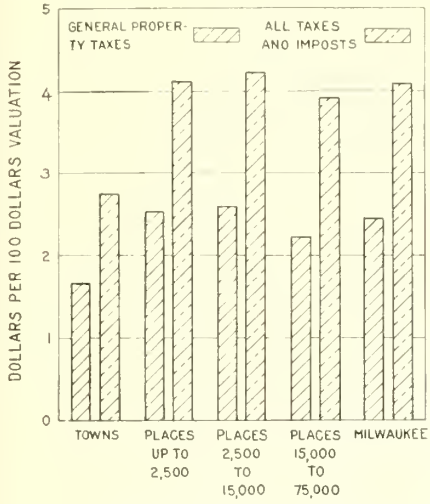
MILEAGE TRAVELED ON THE SEVERAL HIGHWAY SYSTEMS AND THE LOCAL STREETS

The mileage traveled by the average motor vehicle over the various systems of highways and streets in 1930 is shown in table 17. Ownership of cars is classified in the same manner as in table 16. Mileages shown here agree closely with the results of traffic surveys made in other States. The State trunk highway system carried 52.7 per cent of the total motor vehicle traffic. The actual percentage traveled on the State trunk highway system is higher than this, as only travel by Wisconsin cars is included. Travel by out-of-State cars, which apparently use the State trunk highway system almost exclusively, would raise this percentage and lower all other percentages appreciably. Only 13.6 percent of all travel occurred on the county trunk highway system, while 33.7 percent of the travel occurred on the local highways, which include village and city streets. It is interesting to note that 72.8 percent of the travel on local highways occurred on the local streets in cities, while only 23.5 percent occurred on the local roads in towns, and 3.7 percent on the local streets in villages.

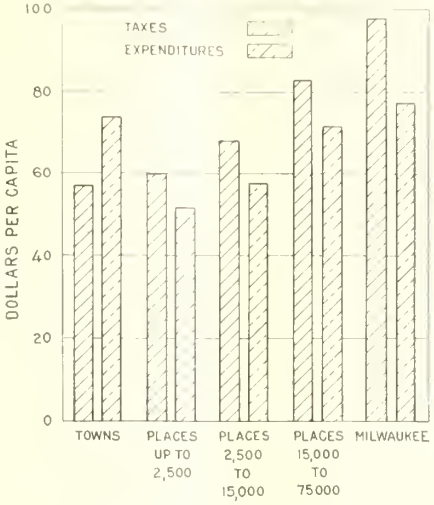
TABLE 17.—*Average mileage traveled on the highway systems and local streets of Wisconsin by Wisconsin motor vehicles in 1930*

Place of ownership and kind of motor vehicle	Average miles traveled						
	Total	State trunk highways	County trunk highways	Local highways and streets			
				Total	Towns	Villages	Cities
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Towns:							
Automobiles	5,583	3,379	1,198	1,006	845	39	122
Motor trucks	6,924	3,567	1,714	1,643	1,353	40	250
Both automobiles and motor trucks	5,751	3,403	1,263	1,085	908	39	138
Percentage [1]	100.0	59.2	22.0	18.8	83.7	3.6	12.7
Villages:							
Automobiles	7,467	4,869	1,210	1,388	600	448	340
Motor trucks	10,147	5,056	2,191	2,900	1,492	1,035	373
Both automobiles and motor trucks	7,895	4,898	1,367	1,630	742	542	346
Percentage [1]	100.0	62.0	17.3	20.7	45.5	33.3	21.2
Cities other than Milwaukee:							
Automobiles	8,671	4,768	856	3,047	327	40	2,680
Motor trucks	11,103	5,122	1,344	4,638	644	61	3,933
Both automobiles and motor trucks	8,999	4,816	921	3,262	370	43	2,849
Percentage [1]	100.0	53.5	10.2	36.3	11.4	1.3	87.3
Milwaukee:							
Automobiles	9,370	3,467	333	5,570	105	30	5,435
Motor trucks	9,027	1,438	232	7,357	41	66	7,250
Both automobiles and motor trucks	9,330	3,225	322	5,783	98	34	5,651
Percentage [1]	100.0	34.6	3.4	62.0	1.7	0.6	97.7
State:							
Automobiles	7,329	3,983	959	2,387	543	81	1,763
Motor trucks	9,002	3,974	1,452	3,576	960	184	2,432
Both automobiles and motor trucks	7,549	3,982	1,024	2,543	598	95	1,850
Percentage [1]	100.0	52.7	13.6	33.7	23.5	3.7	72.8

[1] Percentage figures in columns 2, 3, and 4 give the distribution of motor vehicle travel on the State system, the county system, and the local roads and streets. Percentage figures in columns 5, 6, and 7 give the distribution of local travel on town roads and village and city streets.

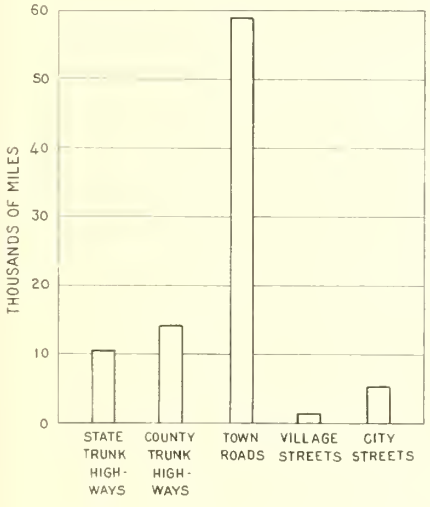


AVERAGE TAX RATES IN THE SEVERAL CLASSES OF GOVERNMENTAL UNIT

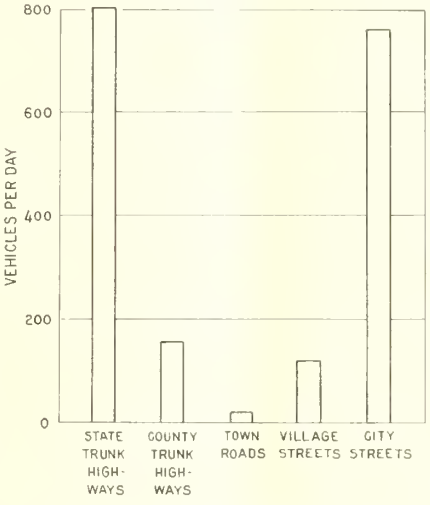


PER CAPITA TAXES AND PER CAPITA EXPENDITURES IN THE SEVERAL CLASSES OF GOVERNMENTAL UNIT

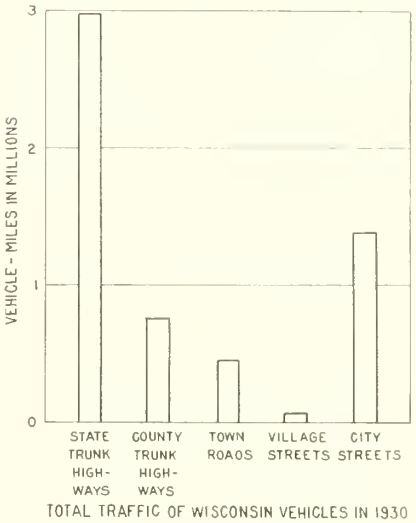
PER CAPITA EXPENDITURES CLASSIFIED BY PURPOSE



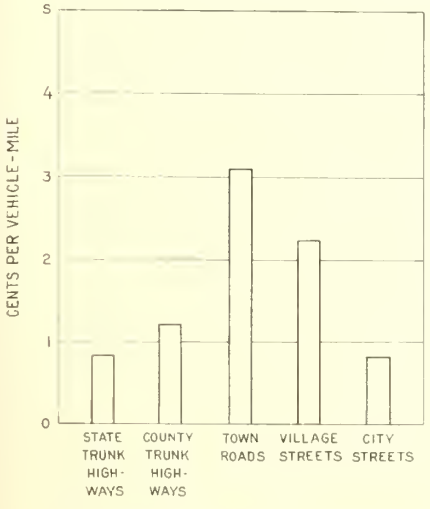
MILEAGE OF HIGHWAY SYSTEMS AND URBAN STREETS



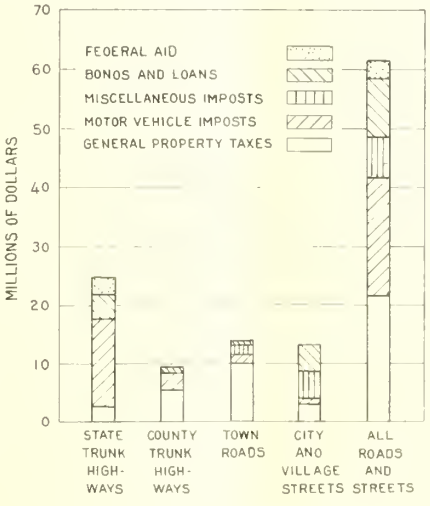
AVERAGE DENSITY OF TRAFFIC ON HIGHWAY SYSTEMS AND URBAN STREETS



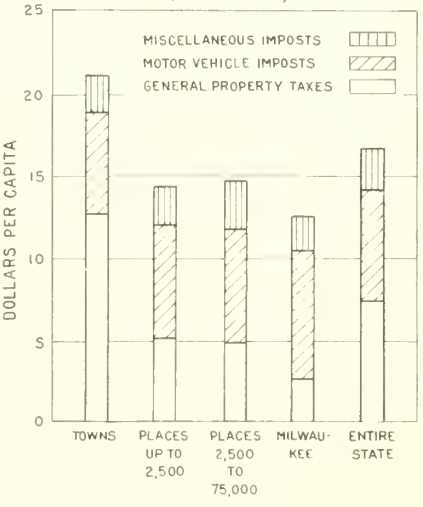
TOTAL TRAFFIC OF WISCONSIN VEHICLES IN 1930 ON HIGHWAY SYSTEMS AND URBAN STREETS (SEE TABLE 21)



EXPENDITURES PER VEHICLE-MILE ON HIGHWAY SYSTEMS AND URBAN STREETS



TOTAL EXPENDITURES ON HIGHWAY SYSTEMS AND URBAN STREETS, SHOWING SOURCES OF FUNDS



PER CAPITA CONTRIBUTIONS BY RESIDENTS OF THE SEVERAL CLASSES OF GOVERNMENTAL UNIT TO TAXES AND IMPOSTS EXPENDED ON ROADS AND STREETS

GRAPHICAL SUMMARY OF ESSENTIAL FACTS DISCLOSED BY SURVEY OF WISCONSIN FINANCES IN 1930

It should also be noted that out of a total of 7,549 miles traveled on the average by all motor vehicles of the State, 1,850 miles, or 24.5 percent of this mileage, was carried by city streets.

The figures contained in this table also indicate that the motor vehicles owned in Milwaukee averaged 9,330 miles, of which 5,651 miles, or 60.5 percent, occurred on city streets; 3,225 miles, or 34.6 percent, occurred on State trunk highways; 322 miles, or 3.4 percent on the county trunk highways; and the remaining 132 miles, or 1.4 percent occurred on town roads and village streets. For motor-vehicles owned in cities other than Milwaukee, almost the same conditions prevail with respect to the use of various highway systems. When city-owned vehicles go outside of their respective cities, they appear to use the State trunk highways much more than county and local roads.

TRAFFIC DISTRIBUTION GROUPED BY COUNTIES

A further distribution of travel and gasoline consumption with respect to the area of the State as a whole was made by grouping the 71 counties according to population per square mile. The classification is shown in table 18. The counties falling into each

TABLE 18.—*Classification of groups of counties according to population per square mile*

Group	Number of counties	Population per square mile	Group	Number of counties	Population per square mile
1[1]	1	3,086	4	19	31 to 45
2	12	76 to 400	5	14	21 to 30
3	10	46 to 75	6	15	Less than 20

[1] Milwaukee County.

group, while not always contiguous, show a rather marked segregation, especially those having less than 20 persons per square mile and those having 75 to 400 persons per square mile.

Table 19 is a representation of travel and gasoline consumption by motor-vehicle owners residing in the towns, villages, and cities of the six county groups described. It discloses the variations in the average mileage traveled which are due more or less to density of population. The general tendency is for the average mileage and gasoline consumed per vehicle to decrease as the population grows less dense. There are several exceptions, but the principal deviation is found in the counties included in group 6, where the population per square mile is the lowest.

TRAVEL OF URBAN AND RURAL VEHICLES DISCUSSED

In table 4 there was given the distribution of registered motor vehicles according to place of ownership. This distribution may be summarized with respect to urban and rural ownership, as follows:

Place of motor vehicle registration	Number registered
In towns	332,379
In incorporated places other than Milwaukee	325,070
In Milwaukee	136,955
Total	794,404

The average mileages given in table 17, when multiplied by the numbers of registered vehicles as listed above, yield the figures on vehicle-mileage by place of ownership which are shown in table 20. Thus we find that 35.7 percent of the travel on the State system in 1930 was originated by rurally owned vehicles, and 64.3

TABLE 19.—*Motor vehicle travel in Wisconsin in 1930 by ownership in the 6 groups of counties* [1]

Item	State totals and averages	Groups of counties: Milwaukee County	Group 2	Group 3	Group 4	Group 5	Group 6
Travel by automobiles:							
Average mileage:							
All automobiles	7,329	9,531	7,071	7,140	6,318	5,847	7,658
Town-owned automobiles	5,583	8,842	5,429	5,521	5,321	5,146	7,053
Village-owned automobiles	7,467	10,645	7,245	8,348	6,765	6,589	9,782
City (except Milwaukee) owned automobiles	8,671	10,352	8,299	9,168	8,563	8,223	8,521
Milwaukee-owned automobiles	9,370	9,370					
Average gallons of gasoline consumed	520	736	485	489	428	391	580
Average motor fuel tax paid	$10.40	$14.72	$9.70	$9.78	$8.56	$7.82	$11.60
Travel by motor trucks:							
Average mileage:							
All trucks	9,002	9,345	10,283	8,423	7,892	7,098	9,980
Town-owned trucks	6,924	10,704	7,778	6,549	6,666	4,884	9,191
Village-owned trucks	10,147	9,891	13,384	7,990	8,509	11,231	8,850
City (except Milwaukee) owned trucks	11,103	11,778	11,631	11,333	9,397	10,163	10,533
Milwaukee-owned trucks	9,027	9,027					
Average gallons of gasoline consumed	851	890	996	749	785	632	833
Average motor fuel tax paid	$17.02	$17.80	$19.92	$14.98	$15.70	$12.64	$16.66
Travel by all motor vehicles:							
Average mileage:							
All vehicles	7,549	9,509	7,497	7,341	6,524	6,072	7,926
Town-owned vehicles	5,751	9,080	5,738	5,679	5,467	5,112	7,232
Village-owned vehicles	7,895	10,502	8,309	8,293	7,042	7,252	9,633
City (except Milwaukee) owned vehicles	8,999	10,427	8,723	9,521	8,701	8,548	9,186
Milwaukee-owned vehicles	9,330	9,330					
Average gallons of gasoline consumed	563	753	552	530	475	429	609
Average motor fuel tax paid	$11.26	$15.06	$11.04	$10.60	$9.50	$8.58	$12.18

[1] This table shows the variations in the mileage and gasoline consumption in the several groups of counties, as indicated by the survey.

TABLE 20.—*Relative contributions of rural and urban motor vehicles to travel on the highway systems and local streets of Wisconsin in 1930*

Annual travel on—	Place of ownership: Towns	Incorporated places other than Milwaukee	Milwaukee	All Wisconsin vehicles	Percentage of total travel
State system:					
Vehicle-miles [1]	1,131.1	1,590.5	441.7	3,163.3	
Percent	35.7	50.3	14.0	100.0	52.7
County system:					
Vehicle-miles [1]	419.8	349.6	44.1	813.5	
Percent	51.6	43.0	5.4	100.0	13.6
Town roads:					
Vehicle-miles [1]	301.8	159.9	13.4	475.1	
Percent	63.5	33.7	2.8	100.0	7.9
Village and city streets:					
Vehicle-miles [1]	58.8	707.7	778.6	1,545.1	
Percent	3.8	45.8	50.4	100.0	25.8
All roads and streets:					
Vehicle-miles [1]	1,911.5	2,807.7	1,277.8	5,997.0	
Percent	31.9	46.8	21.3	100.0	100.0

[1] In millions.

percent by urban vehicles. On the county system the division was approximately equal—51.6 rural and 48.4 urban. On the town roads rural vehicles contributed 63.5 percent of the vehicle-mileage. Traffic in the cities was predominantly urban, town-owned vehicles contributing only 3.8 percent.

TABLE SHOWS RELATION OF TRAVEL AND EXPENDITURES

By using the facts as to travel in Wisconsin developed by the survey of traffic in connection with the known mileage of each system of highways, the known motor-vehicle license fees and gasoline taxes, and the known expenditures, it was possible to construct table 21,

showing many basic relationships between the several highway systems.

TABLE 21.—*Travel and expenditures upon the various highway systems and the local streets in 1930* [1]

Item	Total	State trunk highways	County trunk highways	Local highways and streets		
				Towns	Villages	Cities
Vehicle-miles traveled:						
Total on each (in millions)	5,661.8	2,986.5	768.0	448.5	71.2	1,387.5
Percentage on each	100.0	52.7	13.6	7.9	1.3	24.5
Daily average per mile of highway	173	801	152	21	125	761
Distribution of motor-vehicle revenues:						
If prorated according to vehicle-mileage	$20,025,800	$10,553,600	$2,723,500	$1,585,900	$249,700	$4,913,000
As actually expended	$20,025,800	$14,794,500	$3,100,000	$1,460,200	$38,900	$632,200
Expenditures:						
Total on each	$61,280,400	$24,918,700	$9,449,100	$13,910,900	$1,590,000	$11,411,700
Percentage on each	100.0	40.7	15.4	22.7	2.6	18.6
Expenditures per vehicle-mile on each (in cents)	1.08	0.83	1.23	3.10	2.23	0.82

[1] This table is based upon the facts as to travel ascertained by the survey of traffic, motor-vehicle imposts, and highway expenditures. In making the computations the mileage traveled by motor vehicles from other States and motor busses and regularly routed motor trucks was disregarded. This mileage was traveled almost exclusively on the State trunk highway system. To this extent, all amounts and percentages shown for travel on the State trunk highway system are low in the case of these roads and high in the case of the other highway systems. Earnings shown are based on the assumption that each system of highways earns in proportion to the mileage traveled upon it.

The first division in the table shows the millions of vehicle-miles traveled on each highway system in 1930, the percentage that this vehicle-mileage is of all miles traveled, and the average daily traffic per mile of highway on each system. The State trunk highways, although comprising only 11.4 percent of the total road and street mileage of the State, carried 52.7 percent of the total traffic; the county trunk highways carried 13.6 percent; and the local highways, including town roads, village and city streets, carried 33.7 percent. The city streets, which alone carried a total of 1,387,500,000 miles, accounted for 24.5 percent of the total traffic. The traffic on the State trunk highways and on the city streets constituted 77.2 percent of the total motor-vehicle traffic of the State.

It will be noted that the average daily traffic on the State trunk highway system so computed is 801 per day; on the county trunk highways, 152 per day. The average on town local roads is 21 per day, while on village streets it is 125, and on city streets, 761. The average movement per mile over all streets and highways in the State was 173 vehicles. The State trunk highways carried, on the average, over five times as much traffic per mile as the county trunk highways, and about 40 times as much traffic per mile as the local rural highways.

The next section in the table concerns itself with a hypothetical distribution of the expenditures from motor-vehicle revenues, in proportion to the vehicle-mileage on each system. Immediately below these figures are those showing how the net proceeds of the motor-vehicle imposts were actually distributed to these highway systems in 1930. Considerably more motor-vehicle money was spent on the State trunk highways than would be called for by a distribution according to vehicle-mileage. On the county system and the town roads the figures do not differ greatly, the county roads receiving somewhat more and the town roads somewhat less than the amount indicated in the distribution according to vehicle-mileage. In 1930 the Wisconsin legislature doubled the allowance of motor vehicle money to the towns; so that if the expenditures of a later year were analyzed the town roads would no doubt be shown to receive considerably more than their quota on the basis of traffic. The village and city streets, on the other hand, received relatively little motor-vehicle money, the bulk of the funds being derived from property taxes, including special assessments. The fact that so great a share of the motor-vehicle revenues goes to the State and county trunk highways is in line with the principle that motor vehicle money should be expended very largely on the roads of general service, while revenues from property taxation should be devoted primarily to urban streets and the land-serving local roads.

In the final section of table 21 the expenditures on the several systems, previously given in table 14, are repeated, together with the corresponding percentages and the expenditures per vehicle-mile on each system.

The expenditures on the State highway system were 40.7 percent of the total. The State system carried, as indicated in the first section of the table, 52.7 percent of the total traffic. The difference between the traffic ratio and the expenditure ratio is not very marked in case of the county trunk highways. There the traffic amounted to 13.6 percent of the total traffic, and expenditures constituted 15.4 percent of the total expenditures.

Among local highways and streets, the town roads although carrying only 7.9 percent of all traffic represent expenditures of 22.7 percent of the total. Village streets are less out of line, with 1.3 percent of total traffic and 2.6 percent of expenditure. City streets reverse the traffic-expenditure ratio, with 24.5 percent of the vehicle mileage and 18.6 percent of total expenditures.

The last item, and a very interesting one, shows the number of cents expended on each highway system per vehicle-mile of travel on the system. It will be noted that on all highway systems, the highway cost per mile of travel was 1.08 cents; on the State trunk-highway expenditure it was 0.83 cent; on the county trunk-highway system, 1.23 cents; on the local town highways it was 3.1 cents; on the local village streets, 2.23 cents; and on the local city streets, 0.82 cent. The expenditures per mile of traffic were less on the city streets than on any other highway system, while the State trunk-highway system showed the lowest rate of all rural highways.

In computing this table, the traffic of motor vehicles from outside of Wisconsin, of motor busses, and of motor trucks traveling on regular routes, was not included. Accordingly, the showing for the State trunk highway system, on which practically all of this travel occurs, is less favorable than would have been the case had these figures been included. On the other hand, the showing of all other road systems would have been lowered.

STUDY OF SURVEY DATA YIELDS SIGNIFICANT FACTS REGARDING TAXES, TRAVEL, AND EXPENDITURES

The point is now reached where is it possible to draw together and compare the essential facts developed by the survey, and from them derive certain conclusions regarding the relations existing in Wisconsin in 1930, between the following factors:

1. Taxes paid for highways by the residents of the various local units of government;

2. The funds expended on the several highway systems and the local streets;

3. The relative contributions by motor-vehicle owners in the various local units to the total travel of Wisconsin vehicles in the State;

4. The relative amounts of travel on the several highway systems and the local streets.

The relation between taxes paid in the various local units of government and expenditures on the road systems and local streets of the State may be derived from a study of table 15. Tables 16, 17, and 20 give the necessary information regarding the travel of motor vehicles; and table 21 shows certain relations between travel and expenditures. In the statement of conclusions given below, references are made to the tables upon which each deduction is based.

CONCLUSIONS

1. Rural property pays no tax for urban streets (table 15).

2. Sixty-one percent of the road tax on urban property goes for rural roads; 39 percent for city streets (table 15).

3. Of the total road tax on rural property (table 15):

(*a*) 11.7 percent goes to State trunk highways;

(*b*) 22.6 percent goes to county trunk highways;

(*c*) 65.7 percent goes to local rural (town) roads.

4. Of the total road tax on city property (table 15):

(*a*) 16.7 percent goes to State trunk highways;

(*b*) 32.1 percent goes to county trunk highways;

(*c*) 11.8 percent goes to local rural (town) roads;

(*d*) 39.4 percent goes to city and village streets.

5. Since, of the total property valuation of $5,896,513,000, 37 percent, or $2,181,710,000, was rural, and 63 percent, or $3,714,803,000, was urban (p. 22 and table 15):

(*a*) Expenditures from property taxes for all highways were at the following rates:

Rural—63.3 cents per $100;

Urban—21.2 cents per $100.

(*b*) Expenditures from property taxes for State trunk highways were at the following rates:

Rural—7.4 cents per $100;

Urban—3.5 cents per $100.

(*c*) Expenditures from property taxes for county trunk highways were at the following rates:

Rural—14.3 cents per $100;

Urban—6.8 cents per $100.

(*d*) Expenditures from property taxes for local rural (town) roads were at the following rates:

Rural—41.6 cents per $100;

Urban—2.5 cents per $100.

(*e*) Expenditures from property taxes for city and village streets were at the following rates:

Rural—no tax;

Urban—8.4 cents per $100.

6. Of the total property taxes expended on all roads (tables 15, 20, 21):

(*a*) 13.5 percent was expended on State trunk highways, upon which 52.7 percent of the total travel occurs;

(*b*) 26.0 percent was expended on county trunk highways, upon which 13.6 percent of the total travel occurs;

(*c*) 46.1 percent was expended on local rural (town) roads, upon which 7.9 percent of the total travel occurs;

(*d*) 14.4 percent was expended on city and village streets, upon which 25.8 percent of the total travel occurs.

7. Of the total motor-vehicle imposts expended on all classes of roads and streets (tables 15, 20):

(*a*) Rural motor-vehicle owners (residents of towns) paid 33.9 percent, and travel by these same rural owners made up 31.9 percent of the total travel on all classes of roads and streets;

(*b*) City and village motor-vehicle owners paid 66.1 percent and made up 68.1 percent of the total travel.

RELATIONS ESTABLISHED BETWEEN HIGHWAY TAXES PAID, HIGHWAY EXPENDITURES, AND TRAVEL ON THE VARIOUS SYSTEMS

The total mileage of highways and streets in Wisconsin in 1930 was 89,539. The State highway system consisted of 10,218 miles, divided into 2,345 miles of primary Federal-aid highways, 3,246 miles of secondary Federal-aid highways, and 4,627 miles of other State trunk highways. The county highway system contained 13,827 miles; the local town roads, 58,934 miles; village streets, 1,567 miles; and city streets, 4,993 miles.

The total volume of traffic (exclusive of out-of-State vehicles) on all roads and streets in Wisconsin during the year 1930 was approximately 6 billion vehicle-miles, of which 52.7 percent occurred on the State highway system, 13.6 percent on the county system, 7.9 percent on town roads, and 25.8 percent on village and city streets.

The average density of traffic on the State highway system was 801 vehicles per day; on the county system, 152 vehicles per day; on the town roads, 21 vehicles per day; on village streets, 125 vehicles per day; on city streets, 761 vehicles per day.

Expenditures on the State highway system in 1930 were $24,918,700, or 0.83 cents per vehicle-mile of travel; on the county system, $9,449,100, or 1.23 cents per vehicle-mile; on the town roads, $13,910,900, or 3.10 cents per vehicle-mile; on village streets, $1,590,000, or 2.23 cents per vehicle mile; on city streets, $11,411,700, or 0.82 cents per vehicle-mile.

Of the total property taxes expended on all roads, 13.5 percent was expended on the State highway system; 72.1 percent on county and town roads; and 14.4 percent on village and city streets.

Of the total of all taxes and imposts expended on all roads and streets, rural property and motor-vehicle owners paid 46.9 percent, and travel by rural vehicles made up 31.9 percent of the total travel on all roads and streets; city and village property and motor-vehicle owners paid 53.1 percent, and travel by city and village vehicles made up 68.1 percent of the total travel. These relations between highway taxes paid and travel were no doubt considerably modified by the increase of the gasoline tax from 2 to 4 cents per gallon, which occurred in 1931.

TABLE 22.—*Comparison of taxation and expenditures in Wisconsin in 1930*

$1,000 in taxes [1]			$1,000 in expenditures				
Residents of—	Pay—	In—	Division by purpose [2]	Subdivision by highway system [3]	Source of expenditure [4]	Amount	Percent	Percentage contribution to traffic on system [5]
Towns	$180.22	Property tax, $599.95.	Education, $325.77.	State highways, $123.24.	Taxes paid in:			
Places up to 2,500	51.00				Towns	$32.78	26.60	35.7
Places 2,500 to 15,000	71.45				Villages and cities	51.89	44.73	61.3
Places 15,000 to 75,000	136.91				Federal aid	15.08	12.24	-
Milwaukee	160.37				Proceeds of bonds and loans	20.49	16.63	-
Towns	33.50	Motor-vehicle fees and gasoline tax, $98.84.	Highways, $302.80.	County trunk highways, $46.63.	Taxes paid in:			
Places up to 2,500	10.15				Towns	20.56	44.08	51.6
Places 2,500 to 15,000	11.27				Villages and cities	22.62	48.51	48.4
Places 15,000 to 75,000	22.09				Proceeds of bonds and loans	3.45	7.41	-
Milwaukee	21.83		Public benefit, $309.61.	Town roads, $68.74.	Taxes paid in:			
Towns	79.82	Other miscellaneous taxes, $301.21.			Towns	58.69	85.37	63.5
Places up to 2,500	22.89				Villages and cities	9.36	13.62	36.5
Places 2,500 to 15,000	34.94				Proceeds of bonds and loans	.69	1.01	-
Places 15,000 to 75,000	79.52		Government, $61.82.	Village and city streets, $64.19.	Taxes paid in:			
Milwaukee	84.04				Towns	1.12	1.75	3.8
					Villages and cities	40.95	63.80	96.2
					Proceeds of bonds and loans	22.12	34.45	-

[1] See tables 2, 5, and 6.
[2] See table 10.
[3] See table 14.
[4] See table 15 and p. 29.
[5] Traffic of Wisconsin vehicles only. See table 20.

8. Of the total motor vehicle imposts (tables 15, 20, 21):

(*a*) 73.8 percent was expended on State trunk highways, upon which 52.7 percent of the total travel occurs;

(*b*) 15.5 percent was expended on county trunk highways, upon which 13.6 percent of the total travel occurs;

(*c*) 7.3 percent was expended on local rural (town) roads, upon which 7.9 percent of the total travel occurs;

(*d*) 3.4 percent was expended on city and village streets, upon which 25.8 percent of the total travel occurs.

9. Of all imposts and taxes expended on all roads (tables 15, 20, 21):

(*a*) 36.4 percent was expended on State trunk highways, upon which 52.7 percent of the total travel occurs;

(*b*) 17.9 percent was expended on county trunk highways, upon which 13.6 percent of the total travel occurs;

(*c*) 28.2 percent was expended on local rural (town) roads, upon which 7.9 percent of the total travel occurs;

(*d*) 17.5 percent was expended on city and village streets, upon which 25.8 percent of the total travel occurs.

10. Of the total of all taxes expended on all classes of roads and streets (tables 15, 20):

(*a*) Rural property and motor-vehicle owners paid 46.9 percent, and travel by rural vehicles made up 31.9 percent of the total travel on all classes of roads and streets;

(*b*) City and village property and motor-vehicle owners paid 53.1 percent and city and village vehicles made up 68.1 percent of the total travel.

TAXES AND EXPENDITURES COMPARED

Table 22 shows a comparison between the sources of $1,000 in taxes in 1930 and the manner in which $1,000 was expended in the same year. The data given in this table were obtained directly from, or computed from, tables given previously in the report.

If it were desired to make the tabulation analogous to a true balance sheet of receipts and outlays for the year, it would be necessary to include proceeds from bonds and loans on the left of the double line, and principal payments on the right; and also to make allowance for balances carried over and carried forward. The complete figures necessary for such a presentation were not available; so that expenditures as given cannot be said to balance receipts from taxes. It is believed, however, that table 22 gives a true picture of the relation between tax money received and actual expenditures in the year 1930.

PURPOSES OF TRAVEL

In undertaking the traffic survey, it was felt that it would be of some interest to know about how much of the indicated mileage of automobiles was for pleasure purposes, and how much was for business purposes; and also to get some idea of the travel and touring habits of Wisconsin motor-vehicle owners. This information was obtained from all automobile owners covered by the survey, but only about 40 percent of the schedules were tabulated in making up table 23. The average total mileages in this sample vary somewhat from the average mileages of the whole survey, because in making up the comparatively small sample, any automobile going a most extraordinary mileage or for apparently extraordinary purposes was disregarded.

This tabulation indicates that practically 60 percent of all automobile travel is for business purposes, and 40 percent is for pleasure purposes. Also, that 35 percent of Wisconsin automobile owners drove into adjacent States in 1930, while only 7.5 percent went into adjacent and distant States both. About 43 percent of all Wisconsin automobiles apparently entered some other State in 1930. It is interesting to note that the farmers did by far the lowest percentage of touring in other States, while, as might be expected, residents of Milwaukee

TABLE 23.—*Purposes of travel of 1,720 Wisconsin automobile owners in 1930, with some information as to their travel in other counties and States* [1]

Place of ownership	Reports tabulated	Average miles traveled			Number visiting other States			Average number of other counties visited	Average farthest number of miles away from home in Wisconsin
		Total	For business	For pleasure	Adjacent	Adjacent and distant	Total		
Towns	708	5,575	3,586	1,989	160	18	178	4.9	95
Percent			64.3	35.7	22.6	2.5	25.1		
Places up to 2,500	187	6,995	3,841	3,154	92	13	105	9.5	147
Percent			54.9	45.1	49.2	7.0	56.2		
Places 2,500 to 15,000	195	8,867	4,870	3,997	95	22	117	10.3	167
Percent			54.9	45.1	48.7	11.3	60.0		
Places 15,000 to 75,000	337	8,043	4,559	3,484	155	35	190	8.8	169
Percent			56.7	43.3	46.0	10.4	56.4		
Milwaukee	293	9,567	6,017	3,550	156	41	197	11.4	185
Percent			62.9	37.1	53.2	14.0	67.2		
Total for State	1,720	7,266	4,364	2,902	608	129	737	7.9	139
Percent			60.1	39.9	35.3	7.5	42.8		

[1] This is a tabulation of 40 percent of all reports of travel made in the survey. The same percentage of reports from each group of units of Government was used to produce a properly balanced sample.

did the most. Only 25.1 percent of the owners of motor vehicles residing in towns traveled in other States than Wisconsin; 67.2 percent of the Milwaukee motor-vehicle owners toured in other States.

In order to get some idea of how many counties in Wisconsin the owners visited, that information was also taken. The average motorist visited about eight counties other than his home county. Again, the farmers did by far the least traveling in other counties.

The owner's statement of the greatest number of miles which the automobile traveled in Wisconsin away from its home garage was also recorded. The average touring limit was apparently 139 miles. Again, the farmers did far less touring than any other class, and again Milwaukee showed the larger wanderlust.

This table is believed to present the first figures of this character ever assembled and published. It indicates quite clearly that the residents of Wisconsin did not travel as extensively in 1930 as has been believed to be the custom. Only 7.5 percent, apparently, went on real tours into other States. The percentage visiting adjacent States, is, of course, high. It would probably be much higher in the average State, for Wisconsin has Lake Michigan on the east, the Mississippi and St. Croix Rivers largely bound it on the west, and Lake Superior also interferes with contacts with Canada.

The percentage of the travel for business and for pleasure was ascertained from the original traffic survey sheets. The purposes of travel were so divided on these sheets that it was possible to distinguish the travel as between pleasure and business travel.

It is believed that the tables presented, giving the results of the State-wide survey of highway traffic, offer an accurate indication of the travel of Wisconsin motor vehicles in 1930.

The method followed, involving the use of schedules or questionnaires, may be subject to unfavorable criticism, but it is believed that the results comprise a valuable contribution to the rapidly growing data on highway traffic developed by other studies in other States. Certainly, many relationships previously unknown or merely guessed at have been brought into the field of probable facts.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

FEBRUARY 28,1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal-aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,347.8	$ 4,859,898.25	$ 2,429,949.02	32	155.1	80.0	235.1	$ 866,276.17	$ 433,138.06	11.9	28.1	40.0	$ 3,223,285.46	Alabama
Arizona	1,231.9	2,793,510.18	1,147,693.53	64	122.8	151.7	274.5	18,624.34	3,724.86	3.2		3.2	117,913.53	Arizona
Arkansas	1,932.5	4,006,784.77	1,890,077.95	57	95.5	61.5	157.0	1,151,280.60	575,484.81	91.6		91.6	1,204,475.14	Arkansas
California	2,460.2	9,218,291.90	2,538,247.64	69	161.8	39.6	201.4	1,759,707.13	170,748.11	53.8		53.8	63,814.28	California
Colorado	1,782.4	3,034,039.61	1,362,561.88	57	115.8	34.0	149.8	776,944.26	349,624.85	52.3		52.3	494,735.72	Colorado
Connecticut	291.7	4,570,893.92	1,834,805.50	63	51.5		51.5	192,642.80	43,559.36	2.5		2.5	68,231.84	Connecticut
Delaware	369.7	675,082.15	123,212.72	54	18.6	21.9	40.5	244,671.10	53,012.72	10.4		10.4		Delaware
Florida	661.0	6,040,733.90	2,884,225.70	59	168.1		168.1	458,404.35	229,202.17	.8		.8	1,072,543.91	Florida
Georgia	3,152.5	6,069,204.42	2,484,736.18	59	172.7	198.5	371.2	206,801.14	93,060.53	3.7	11.0	14.7	28,687.83	Georgia
Idaho	1,531.0	2,705,139.03	1,032,933.28	67	121.9	130.6	252.5	233,569.15	83,413.50	18.0		18.0	87,609.68	Idaho
Illinois	3,068.8	21,182,457.74	8,213,217.18	79	650.4	38.2	688.6	1,178,984.89	487,319.49	48.1		48.1	20,310.48	Illinois
Indiana	2,066.7	7,331,746.19	3,300,110.69	89	270.6		270.6	2,483,461.38	259,094.38	117.1	5.3	122.4	47,304.24	Indiana
Iowa	3,515.5	5,713,863.68	1,005,785.38	87	286.1	40.0	326.1	29,981.52	11,160.00		1.2	1.2	18,402.48	Iowa
Kansas	3,806.8	4,493,120.00	1,664,371.55	70	293.6	113.9	407.5	1,325,848.95	226,029.42	81.1	23.8	104.9	242,408.59	Kansas
Kentucky	1,888.9	4,729,835.34	1,712,240.85	53	208.3	118.8	327.1	5,128.59	2,154.00				153,237.29	Kentucky
Louisiana	1,503.7	7,348,393.34	3,211,146.73	54	55.1	26.3	81.4							Louisiana
Maine	868.9	2,333,995.17	497,645.93	67	70.3	.3	70.6	92,667.35	30,843.45	.2	1.9	2.1	83,173.45	Maine
Maryland	827.5	1,438,599.92	446,696.63	86	65.7	2.5	68.2	354,830.40	22,640.00	7.3		7.3	33,778.68	Maryland
Massachusetts	870.5	4,305,603.05	1,086,714.88	59	59.8	4.9	64.7	158,206.29	30,000.00	3.4		3.4	30,738.38	Massachusetts
Michigan	2,293.8	7,391,967.13	2,982,824.95	65	311.5	103.0	414.5	1,158,975.91	396,990.00	69.4	1.4	70.8	53,252.74	Michigan
Minnesota	4,308.6	4,645,947.47	133,628.91	84	149.7	139.4	289.1	114,649.27	100.00	1.3	.4	1.7	81,585.43	Minnesota
Mississippi	1,823.9	6,610,234.84	3,261,936.81	55	226.3	78.9	305.2	515,350.51	257,675.22	17.7	2.5	20.2	3,220,860.41	Mississippi
Missouri	3,187.8	3,004,002.98	632,312.18	70	129.2	11.0	140.2	2,052,193.31	371,194.71	103.7		103.7	185,115.24	Missouri
Montana	2,730.8	6,920,347.56	3,880,467.26	57	508.3	288.2	796.5	666,048.38	374,331.22	78.6	11.8	90.4	231,482.98	Montana
Nebraska	4,269.3	5,769,048.69	2,713,854.04	43	165.6	136.7	302.3	169,336.37	76,264.76	10.4	2.4	12.8	237,279.76	Nebraska
Nevada	1,353.3	1,566,696.03	619,307.41	86	29.7	105.9	135.6	482,542.12	14,435.12	29.6	13.0	42.6	124,455.24	Nevada
New Hampshire	434.8	795,156.51	336,481.31	69	20.2	3.4	23.6	144,280.17	57,712.07	1.4	.6	2.0	90,145.58	New Hampshire
New Jersey	629.2	6,215,931.90	2,132,266.20	65	64.3		64.3						65,234.22	New Jersey
New Mexico	2,248.8	2,765,649.43	1,211,028.87	65	161.5	77.8	239.3	569,300.04	238,648.57	15.4	19.0	34.4	104,379.99	New Mexico
New York	3,497.0	17,395,729.40	5,403,114.73	49	466.7	28.4	495.1	2,333,150.65	805,290.00	46.5	3.6	50.1		New York
North Carolina	2,259.3	4,333,366.04	2,163,071.83	40	487.3	23.6	510.9	918,782.67	460,853.84	131.8	.5	132.3	1,370,211.81	North Carolina
North Dakota	5,273.3	4,425,984.33	1,846,134.77	53	376.5	383.7	760.2	912,193.31	297,291.16	68.7	253.2	321.9	293,980.19	North Dakota
Ohio	2,999.9	8,855,588.09	2,471,433.49	70	203.5	69.8	273.3	1,118,304.00	321,755.27	24.2	6.3	30.5	98,435.05	Ohio
Oklahoma	2,438.3	3,889,265.46	1,675,825.51	74	158.3	65.2	223.5	2,436,370.92	502,689.38	165.2	18.3	183.5	362,652.89	Oklahoma
Oregon	1,592.5	3,972,568.18	1,639,420.09	60	115.6	94.6	210.2	481,138.53	170,712.28	14.2	.4	14.6	87,867.71	Oregon
Pennsylvania	3,166.9	11,949,794.44	3,490,400.55	55	433.3	14.0	447.3	771,311.22	294,259.43	25.0		25.0	34,232.89	Pennsylvania
Rhode Island	271.6	1,071,153.47	361,357.27	25	22.5	4.5	27.0						31,384.36	Rhode Island
South Carolina	1,957.4	3,513,090.79	1,326,892.73	57	162.5	158.3	320.8	77,398.09	10,787.67		3.4	3.4	222.17	South Carolina
South Dakota	4,254.2	3,973,083.59	1,689,619.10	74	259.7	203.5	463.2	167,351.17	40,466.93	33.7		33.7	65,951.50	South Dakota
Tennessee	1,684.7	5,179,156.96	2,588,857.51	32	163.6	61.6	225.2	394,222.62	197,111.30	14.0	5.7	19.7	560,086.44	Tennessee
Texas	7,808.7	18,592,366.15	7,269,032.98	63	738.0	475.1	1,213.1	3,391,700.80	892,018.23	187.2	87.1	274.3		Texas
Utah	1,254.6	1,687,737.42	696,942.06	70	96.4	77.9	174.3	351,885.56	106,059.22	27.0	15.5	42.5	205,039.73	Utah
Vermont	385.8	508,094.30	58,754.29	62	26.6		26.6						16,037.73	Vermont
Virginia	1,936.8	4,637,691.97	2,045,209.12	54	216.6	53.1	269.7	725,747.16	362,723.48	29.5	17.4	46.9	185,219.21	Virginia
Washington	1,331.3	2,454,333.87	787,612.10	44	100.9	1.1	102.0	814,177.77	187,100.00	16.7	8.9	25.6	79,492.38	Washington
West Virginia	913.0	3,594,236.97	1,450,952.71	67	125.0	8.5	133.5	109,146.87	43,546.87	.2		.2	44,851.29	West Virginia
Wisconsin	2,745.2	6,130,555.98	1,186,806.56	75	159.3	102.0	261.3	92,061.50	26,500.00	.1	.9	1.0	153,605.97	Wisconsin
Wyoming	2,052.4	3,163,889.56	1,126,740.50	81	265.1	179.2	444.3	399,476.10	39,588.60	39.0	14.5	53.5	26,890.37	Wyoming
Hawaii	78.4	2,316,038.32	1,238,558.23	50	62.3		62.3	164,527.74	164,527.74	3.7		3.7	745,364.73	Hawaii
TOTALS	105,411.6	260,184,900.39	97,337,267.29	61	9,549.7	4,011.1	13,560.8	33,049,658.17	9,815,842.78	1,659.6	558.1	2,217.7	15,745,968.99	TOTALS
Construction completed		144,060,000	59,497,000											
Balance uncompleted		116,125,000	37,840,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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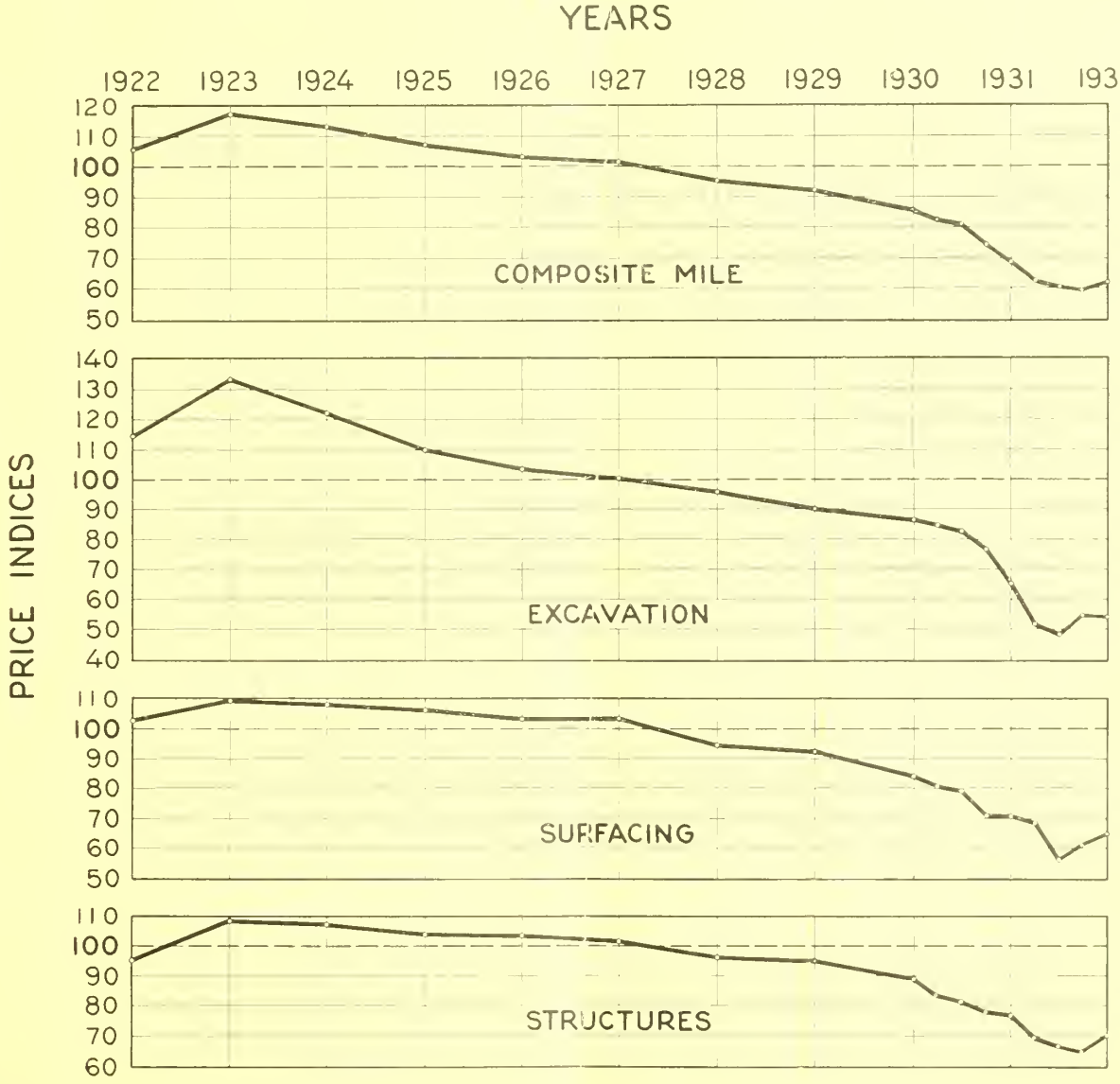

TREND OF HIGHWAY CONSTRUCTION COST

BASE 1925-1929 FEDERAL-AID COSTS

For sale by the Superintendent of Documents, Washington, D.C. - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

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BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 3 May 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

THE ILLINOIS FINANCIAL SURVEY

Digest of a Survey of the Finances of Illinois in 1930, with Special Reference to Highways, Conducted by the Bureau of Public Roads and the University of Wisconsin

THIS STUDY of highway finance in Illinois is one of a series conducted by the United States Bureau of Public Roads in cooperation with the University of Wisconsin and various State highway departments, involving expenditures for highway purposes and the incidence of taxes for these and other purposes. The Illinois survey is based on the calendar year 1930.[1] The following summary of the comprehensive report made by the investigators has been prepared by the Division of Highway Transport of the Bureau in order to make the results available in concise form. The arrangement of material follows that of the Wisconsin summary for greater ease of comparison.[2]

The calendar year 1930 was selected for the study because of its being a United States census year, as well as the latest year for which data from Cook County could be obtained. It is also the same year for which the similar studies in Wisconsin and Michigan were made.

To classify the incidence of taxation, the State was divided into groups of governmental units and into classes of residence. To distribute highway expenditures, the highway classifications in use in the State were adopted.

Division into groups of governmental units and classes of residence was as follows:

The State government.

The counties:

Group 1.—All counties with a population of over 400 persons per square mile in 1930.

Group 2.—All counties with a population of from 75 to 400 persons per square mile in 1930.

Group 3.—All counties with a population of from 45 to 74 persons per square mile in 1930.

Group 4.—All counties with a population of from 30 to 44 persons per square mile in 1930.

Group 5.—All counties with a population of from 20 to 29 persons per square mile in 1930.

The municipalities, or local government units:

Townships (embracing all rural areas outside of incorporated municipalities).[3]

Incorporated places having a population of 2,500 or less.

Incorporated places having a population of 2,500 to 15,000.

Incorporated places having a population of 15,000 to 75,000.

Incorporated places having a population of 75,000 to 400,000.

The city of Chicago.

The purposes of the investigation were as follows: (1) To study and analyze the direct and indirect highway receipts and disbursements of the State and of the counties, townships, municipalities, and other political subdivisions thereof; (2) to develop facts and information showing the effect of highway and related taxes on property of all kinds; and (3) to develop the total amount of revenues raised for all other governmental purposes of the State, counties, and local units as compared with the revenues raised for highways, bridges, and streets.

Illinois was selected as a field for study partly because of the representative character of its centralized highway development and partly because the facts as to receipts, expenditures, and debts of the State had never before been assembled. It was believed of value, therefore, to present information relative to the highway situation in Illinois, and also to obtain general financial statistics for use in intelligently planning future Illinois highway programs.

It is natural that these exhaustive investigations should reveal many incidental facts. This has been the case in Illinois, where certain conditions have been noted which appear worthy of mention: (1) There was apparent, quite generally, evidence of conscientious efforts on the part of most local fiscal agents in all branches of the State government to carry on their work in a satisfactory and cooperative way under difficult and complex conditions which seemed to the investigator to be most discouraging; (2) the fiscal system and records of the State-wide organization of counties and local units are cumbersome and unscientific; (3) the State has no central accounting control of any kind: (4) the taxing authority is excessively subdivided; and (5) the existing organization automatically creates unduly heavy overhead for local road administration.

The reader should bear in mind that the present study relates only to the year 1930. Students of taxation problems will readily realize that the situation here portrayed differs somewhat from that existing today in Illinois and Chicago. The value of this study, however, lies in its analysis of fundamental relationships between taxes and expenditures, and is not diminished by temporary and abnormal conditions affecting tax collections.

In considering the financial data for Illinois, care must be used to make proper allowance for the great influence exerted by the city of Chicago upon all trends and averages. In this city were concentrated over 44 percent of the population, over 31 percent of all motor vehicles, and over 45 percent of the assessed valuation of the entire State of Illinois. Upon taxpayers of this city

[1] The investigation was directed by Henry R. Trumbower, professor of economics at the University of Wisconsin and economist for the Bureau of Public Roads, assisted by the late A. R. Hirst, engineer, and H. R. Triggs, statistician. The Division of Highways, Illinois Department of Public Works and Buildings, cooperated in making the facts and data available. The full report was recently published by the State of Illinois, under the title "Economic Survey of Illinois."

[2] For results of the Wisconsin survey, see the April 1933 issue of PUBLIC ROADS

[3] There are in Illinois incorporated cities, villages, and towns. All such municipalities are included in the classification of incorporated places.

were levied more than 55 percent of all taxes levied within the State; and more than 52 percent of the total public expenditures in Illinois in 1930 was made in Chicago.

GOVERNMENTAL SYSTEM DESCRIBED

The State Government is of the usual type, with a General Assembly composed of 2 houses, a Governor and other State elective offices, 9 executive departments and other appointive bodies.

There is a State tax commission the duties of which are to fix the valuation of railroads and certain property of corporations, and generally to supervise local property taxation.

There are two distinct types of county government—17 counties in the southern part of the State have a county unit system and 85 counties have the township form of organization; in Cook County there is a special form of county organization.

Local government is complex, with many concurrent overlapping governmental bodies. The taxpayer may find himself subject to concurrent property taxes levied by the following agencies: State, county, township, village or city, high-school district, common-school district, sanitary district, forest-preserve district, park district, and mosquito-abatement district, or 10 governmental agencies. And, in addition, he may be subject to special assessments for streets, sidewalks and sewers. Each of these agencies has its own set of officers, its own records, and an authority to levy taxes and incur indebtedness. In this study it has not been the practice to divide the units of government further than the townships and incorporated places. Other minor governmental areas have been treated as parts of the larger units containing them.

POPULATION AND ASSESSED VALUATION

In 1930 the population of Illinois was 7,630,654. Concentration of population was primarily urban—6,286,995 persons being inhabitants of incorporated places and 1,343,659 being residents of unincorporated territory. These urban populations tend to concentrate in metropolitan areas, as shown by the fact that 4,486,441 persons live in cities of over 25,000. Of these, 3,376,438 live in the city of Chicago.

The assessed valuation of all taxable property in Illinois in 1930 was $7,718,657,700. Twenty-six percent of the assessed valuation was in the townships; 6 percent in incorporated places having a population up to 2,500; 8.8 percent in incorporated places having a population of 2,500 to 15,000; 11.3 percent in places having a population of 15,000 to 75,000; 2.2 percent in places having a population of 75,000 to 400,000; and 45.7 percent was in Chicago. Thus, 26 percent of the valuation was in the rural areas, and 74 percent in the urban areas.

So far as could be ascertained, real estate in Illinois was assessed in 1930 at from 30 to 40 percent of its actual value, and personal property at only nominal values.

The report on Illinois indicates that many of the relationships found to exist in Wisconsin are duplicated. Conspicuous among these are the substantial contributions to rural areas from the various municipal groups. There is an indication, however, that the large metropolitan centers such as Milwaukee in Wisconsin, Chicago in Illinois, and Detroit in Michigan are not only unique in the State but sometimes have a tendency to alter very substantially, or even to reverse, observed tendencies.

HIGHWAY SYSTEMS IN 1930

In 1930 there were three systems of rural highways in Illinois.

Of the rural highway system, 10,098 miles, or 10.4 percent of the total, comprise the State primary system. This system is under the jurisdiction of the Illinois Department of Public Works and Buildings, Division of Highways. It is a system designed as a State trunk system and is for the most part well paved.

Seventeen thousand three hundred sixty-nine miles, or 17.9 percent of the total, comprise the county highway system. This system is under the joint jurisdiction of the counties and the State Department of Public Works and Buildings. It serves principally county travel. The larger part of this system consists of earth roads.

Seventy-one and seven tenths percent of the total rural highways of the State are local roads under the jurisdiction of local authorities. It is interesting to note that these locally administered roads comprise a mileage nearly seven times as extensive as that of the system under direct control of the Division of Highways. With the exception of a small percentage of gravel, earth roads comprise practically all of this system. Table 1 classifies the rural mileages by highway systems and by type of surfacing. The financing of each of these types will be discussed in later pages.

The total rural highway mileage of the State amounts to 97,234 miles, or an average of 1.735 miles per square mile of territory. This is the mileage as traveled in 1930 and may vary somewhat from other mileages based on systems as officially laid out or designated.

TABLE 1.—*Mileage and surfacing of rural highways in 1930*

System	Mileage and type of surfacing								Total		
	Concrete		Bituminous macadam		Stone, gravel, etc.		Earth				
	Miles	Percent	Miles	Percent	Miles	Percent	Miles	Percent	Miles	Percent	Percentage of total
State	7,683	76.1	26	0.3	121	1.2	2,268	22.4	10,098	100.0	10.4
County	1,453	8.4	130	.7	4,726	27.2	11,060	63.7	17,369	100.0	17.9
Township	83	.1	90	.1	11,190	16.1	58,404	83.7	69,767	100.0	71.7
Total	9,219	9.5	246	.2	16,037	16.5	71,732	73.8	97,234	100.0	100.0

At the end of 1930 there were 25,502 miles of surfaced rural highways in Illinois, or 26.2 percent of all rural highways. On the State system, 77.6 percent were surfaced; on the county system, 36.3 percent; and on the local roads, 16.3 percent.

Of the total rural highway mileage in the State, 73.8 percent is earth.

The Illinois State highway system, which is under the control of the Division of Highways, and for which the department is directly responsible for construction and maintenance, is divided into 2,724 miles known as the primary Federal-aid system; 4,252 miles called the secondary Federal-aid system; and the remaining 3,122 miles are classed as other State highways. The entire system is commonly known as the State primary system, or the State bond issue road system.

The funds for this State primary system are derived from State imposts upon motor vehicles, from Federal

aid and from bond issues. As the bond issues are being retired out of the motor-vehicle imposts, in the final analysis the entire program will be carried by these imposts, plus the Federal aid.

Few facts are available as to city and village streets in the State of Illinois. Nothing is known as to their total mileage or as to their types of surface. It is known that out of the $166,298,900 expended on all highways in Illinois in 1930, $100,100,200, or two thirds of the total of all highway expenditures of the year, were local expenditures on municipal streets. Of this amount, $75,873,300 were expended by the city of Chicago alone.

METHOD OF OBTAINING DATA DESCRIBED

Despite difficulties which at first seemed insurmountable—including the absence of a central assembly of financial and tax statistics, the scarcity of public reports of counties and municipalities, and the diversified and complicated systems of municipal accounting existing in Illinois—preliminary visits to some of the leading counties and cities showed that much information was available.

It was decided, therefore, to obtain the primary data for the counties and the larger cities by a personal visit to each, for the purpose of analyzing and collecting the material on the premises. The result is that on probably half of the counties and the majority of the larger cities, accurate detailed data have been secured from audit reports. On the majority of the remaining counties and cities, good information has been obtained which was compiled directly from the county records. Of the balance of the counties and cities which were for the most part small with poor records, the best possible estimates were made from all of the information available on the premises, plus general information obtained by interviewing the officials.

Encouraged by the results obtained from the counties, the investigators decided to attempt a simple financial questionnaire for the townships, villages, and minor cities. The results exceeded expectations, over 1,000 good replies being received. These for the most part indicate a conscientious, intelligent effort on the part of the local clerks to give the data correctly. Through averages and ratios based on these replies, a correct picture of the minor local Illinois governmental units can be drawn.

To obtain figures on such matters as mileage traveled by motor vehicles, gasoline consumed, and taxes paid, questionnaires were sent to many thousands of motor vehicle owners. Additional data were secured through analysis of a large sample of car registrations. Further details on these points will be found in the section on "Motor-Vehicle Registration, Taxes, and Travel." (See p. 41.)

TAXES LEVIED AND INCIDENCE OF TAXATION

Table 2 gives the total taxes levied for all purposes in Illinois in 1930 by the State government, the counties, and the several classes of local government unit. The importance of Chicago in the financial set-up of the State is emphasized by the fact that of all taxes levied in Illinois in 1930, 41.2 percent were imposed by the city of Chicago.

Since all State and county imposts and taxes are ultimately paid by taxpayers in townships and incorporated places, the amounts shown in the above table may be restated to show their distribution as ultimately paid. This distribution is shown in table 3. Here again it is found that 55.2 percent of all the charges in 1930 were levied on taxpayers in Chicago.

ILLINOIS TAXES IN 1930

Total taxes and imposts levied for all purposes in Illinois in 1930 were $582,407,100. Taxation of general property totaled $427,317,600; all motor-vehicle imposts, including gasoline tax, $54,149,100; other sources, including insurance and inheritance taxes, valuation tax on railroads, corporation imposts, certain local imposts, and miscellaneous State revenues, $100,940,400.

Of the total taxes and imposts levied, rural taxpayers were charged with $71,516,100, or 12.3 percent; those in incorporated places having a population up to 2,500, $38,011,300, or 6.5 percent; in places of 2,500 to 15,000 population, $65,583,100, or 11.2 percent; in places of 15,000 to 75,000 population, $73,745,800, or 12.7 percent; in places of 75,000 to 400,000 population, $12,068,300, or 2.1 percent; in Chicago, $321,482,500, or 55.2 percent.

The average actual tax rates on general property, per $100 valuation, were as follows: Rural, $2.81; incorporated places up to 2,500 population, $4.59; places 2,500 to 15,000 population, $6.20; places 15,000 to 75,000 population, $5.71; places 75,000 to 400,000 population, $4.74; Chicago, $7.08.

Rural motor vehicles owners paid in license fees and gasoline taxes an average of $21.77; those resident in places up to 2,500 population, $25.81; in places 2,500 to 15,000 population, $28.19; in places 15,000 to 75,000 population, $30.31; in places 75,000 to 400,000 population, $21.53; in Chicago, $31.41.

TABLE 2.—*Total of all State and local charges imposed for all purposes by the State and its subdivisions for collection in 1930*

Imposed by—	Amount	Percent	Per capita
State	$136,227,300	23.4	$17.85
County	45,208,900	7.8	5.92
Townships	40,407,800	6.9	30.07
Places up to 2,500	22,328,600	3.8	34.28
Places 2,500 to 15,000	44,079,700	7.6	36.41
Places 15,000 to 75,000	47,140,200	8.1	12.14
Places 75,000 to 400,000	7,180,600	1.2	37.63
Chicago	239,834,000	41.2	71.03
Total	582,407,100	100.0	76.32

TABLE 3.—*Incidence of all State and local charges imposed in 1930*

Payable by taxpayers in—	Amount	Percent	Per capita
Townships	$71,516,100	12.3	$53.22
Places up to 2,500	38,011,300	6.5	58.37
Places 2,500 to 15,000	65,583,100	11.2	69.05
Places 15,000 to 75,000	73,745,800	12.7	65.92
Places 75,000 to 400,000	12,068,300	2.1	63.24
Chicago	321,482,500	55.2	95.21
Total	582,407,100	100.0	76.32

The sources of total Illinois revenue for all purposes in 1930 are shown in table 4. It is to be noted that in these tabulations there have been included both the taxes levied but not fully collected, principally general property taxes; and taxes actually collected, such as motor-vehicle, corporation, inheritance, and other imposts. (See p. 45.)

TABLE 4.—*Sources of revenue by type of tax or revenue*

Type of tax	Amount	Per-cent	Per capita
Ad valorem general taxes	$427,317,600	73.4	$56.00
Ad valorem railroad taxes	31,272,400	5.4	4.10
Other railroad taxes	2,478,900	.4	.32
State motor-vehicle imposts	46,974,600	8.1	6.15
Local vehicle imposts	7,174,500	1.2	.94
Other local imposts	41,570,400	7.1	5.45
Inheritance taxes	12,802,100	2.2	1.68
Insurance taxes	6,342,000	1.1	.83
Corporation imposts	4,088,600	.7	.54
Miscellaneous State revenues	2,386,600	.4	.31
Total	582,407,100	100.0	76.32
RECAPITULATION			
General property taxes	$427,317,600	73.4	$56.00
Railroad taxes	33,751,300	5.8	4.42
Motor-vehicle imposts	54,149,100	9.3	7.09
Other local receipts	41,570,400	7.1	5.45
Other State receipts	25,618,700	4.4	3.36
Total	582,407,100	100.0	76.32

TAXATION DATA SUMMARIZED

In order to summarize the total revenues of the State as shown by table 4, another distribution of these revenues to show their incidence by general types of taxes and by classes of places in which they are paid is given in table 5. The item "other taxes" includes imposts on railroad property, inheritance taxes, insurance taxes, corporation imposts, and miscellaneous State and local imposts. The ad valorem railroad taxes should properly be classified under general property taxes, but no break-down was available on this item taken alone.

TABLE 5.—*Incidence and classification of all taxes levied in 1930*

Payable by taxpayers in—	General property taxes		Imposts on motor vehicles [1]		Other taxes		All taxes	
	Amount	Per-cent	Amount	Per-cent	Amount	Per-cent	Amount	Per-cent
Townships	$56,409,700	13.2	$4,828,600	8.9	$10,277,800	10.2	$71,516,100	12.3
Places up to 2,500	21,263,500	5.0	7,255,100	13.4	9,492,700	9.4	38,011,300	6.5
Places 2,500 to 15,000	41,953,700	9.8	8,543,600	15.8	15,085,800	14.9	65,583,100	11.3
Places 15,000 to 75,000	49,698,600	11.6	9,900,300	18.3	14,146,900	14.0	73,745,800	12.6
Places 75,000 to 400,000	8,187,200	1.9	1,911,800	3.5	1,969,300	2.0	12,068,300	2.1
Chicago	249,804,900	58.5	21,709,700	40.1	49,967,900	49.5	321,482,500	55.2
		100.0		100.0		100.0		100.0
Total	427,317,600	73.4	54,149,100	9.3	100,940,400	17.3	582,407,100	100.0

[1] Includes license fees, $18,447,200; gasoline tax, $28,246,700; oil-inspection fees, $280,700; and wheel taxes, $7,174,500. (See pp. 41 to 44 for detailed discussion.)

Table 6 gives, in the first column of figures, the tax rates per $100 valuation required to meet the total charges levied upon general property in each of the several classes of local government unit. For purposes of comparison, the second column gives the rates which would have been required if all taxation had been levied upon general property; and the last column gives the percentages which general property taxes were of all levies.

TABLE 6.—*General property tax rates per $100 valuation in 1930, and their relation to the total of all imposts*

Unit of government in which taxes were payable	Actual tax rate on general property tax as levied	Tax rate needed to raise all taxes by general property tax levies	Percentage that general property taxes are of all taxes and imposts
Townships	$2.81	$3.56	78.9
Places up to 2,500	4.59	8.20	55.9
Places 2,500 to 15,000	6.20	9.69	64.0
Places 15,000 to 75,000	5.71	8.47	67.4
Places 75,000 to 400,000	4.74	6.98	67.8
Chicago	7.08	9.12	77.7
Average for State	5.54	7.55	73.4

STREET AND HIGHWAY REVENUES OBTAINED FROM GENERAL PROPERTY AND MOTOR-VEHICLE TAXES

The amount of $150,266,700 was levied in the State of Illinois in 1930 for street and highway purposes. Of this sum, the charges imposed directly against motor vehicles and their owners and operators was somewhat over one third of the total, the balance being charges against property. Table 7 shows these facts in detail.

TABLE 7.—*Taxation imposed for street and highway purposes in 1930*

Type of tax	Amount	Per-cent	Per capita
On general property:			
County	$7,076,200	4.7	$0.93
Local	29,122,800	19.4	3.81
Special	56,260,200	37.4	7.37
Railroad	3,658,400	2.4	.48
Total	96,117,600	63.9	12.59
On motor vehicles:			
License fees	18,447,200	12.3	2.42
Fuel tax	28,246,700	18.8	3.70
Oil inspection fees	280,700	.2	.04
Privilege or "wheel" taxes (local)	7,174,500	4.8	.94
Total	54,149,100	36.1	7.10
Grand total	150,266,700	100.0	19.69

TABLE 8.—*Incidence of taxes imposed for street and highway purposes in 1930, classified by groups of places*

Payable by taxpayers and motor-vehicle owners in—	Amount	Per-cent	Per capita
Townships	$16,996,900	11.3	$12.65
Places up to 2,500	15,059,300	10.0	23.12
Places 2,500 to 15,000	21,479,300	14.3	22.62
Places 15,000 to 75,000	22,114,800	14.7	19.77
Places 75,000 to 400,000	3,702,800	2.5	19.40
Chicago	70,913,600	47.2	21.00
Total	150,266,700	100.0	19.69

Table 8 shows the distribution of the street and highway levies as amounts paid (or payable) by taxpayers and motor-vehicle owners in the several classes of local government unit. The amount given for each class of place is the sum of all taxes imposed for highway purposes, including local privilege taxes and portions of the local general property tax levies, local special assessments, and local railroad taxes [4]; that portion of the

[4] Out of a total of $31,272,400 in local railroad taxes, $3,658,400 were levies for local roads and streets.

county property taxes allocated to the communities on the basis of relative property values; and the amount of State gasoline and vehicle fees contributed by motor vehicle owners in these groups of places as determined by facts ascertained in this study.

For the cities, this includes the contributions for local street improvements as well as the urban share of the rural highway program. The same facts classified by county groups instead of groups of places are shown in table 9.

TABLE 9.—*Incidence of taxes specifically imposed for street and highway purposes in 1930, classified by groups of counties*

Payable by taxpayers and motor-vehicle owners in—	Amount	Per-cent	Per capita
County group:			
1	$87,512,900	58.2	$21.98
2	29,217,100	19.5	17.61
3	18,888,100	12.6	17.31
4	12,057,800	8.0	16.83
5	2,590,800	1.7	14.27
Total	150,266,700	100.0	19.69

MOTOR-VEHICLE REGISTRATION, TAXES, AND TRAVEL

Registrations.—There were 1,642,628 motor vehicles of all types registered in the State of Illinois in 1930, or 1 for every 4.6 persons. The ratio of passenger cars was 1 to every 5.3 persons, and of motor trucks and busses 1 to every 37.1 persons.

In order to allocate motor vehicles among the local units of government, a sample of 50,000 motor-vehicle registrations was tabulated, and the situs of vehicles whose location was uncertain was verified by correspondence. On the basis of the distribution shown by this sample, an approximate allocation was made, by place of ownership, of all registered motor vehicles, among the several county groups and the classes of local unit. This distribution is shown in table 10. The data are subdivided between passenger automobiles on the one hand and trucks and busses on the other.

TABLE 10.—*Number of registered automobiles and trucks and busses in 1930, distributed by place of ownership* [1]

County group	Place of ownership						Total	Percentage of State total
	Townships	Places up to 2,500	Places 2,500 to 15,000	Places 15,000 to 75,000	Places 75,000 to 400,000	Chicago		
Group 1:								
Automobiles	5,134	10,268	35,937	71,875		447,219	570,433	
Trucks and busses	2,003	1,233	4,161	6,010		63,644	77,051	
Total	7,137	11,501	40,098	77,885		510,863	647,484	39.4
Group 2:								
Automobiles	36,509	45,649	77,287	138,879	17,525		315,849	
Trucks and busses	11,346	4,992	8,447	21,897	6,671		53,353	
Total	47,855	50,641	85,734	160,776	54,196		399,202	24.3
Group 3:								
Automobiles	59,761	77,930	87,348	54,800			279,839	
Trucks and busses	12,906	6,808	11,458	7,356			38,528	
Total	72,667	84,738	98,806	62,156			318,367	19.4
Group 4:								
Automobiles	59,796	92,671	43,170				195,637	
Trucks and busses	14,661	9,497	6,673				30,831	
Total	74,457	102,168	49,843				226,468	13.8
Group 5:								
Automobiles	15,605	24,953	4,500				45,058	
Trucks and busses	2,918	2,763	368				6,049	
Total	18,523	27,716	4,868				51,107	3.1
Total for State:								
Automobiles	176,805	251,471	248,242	265,554	47,525	447,219	1,436,816	87.5
Trucks and busses	43,834	25,293	31,107	35,263	6,671	63,644	205,812	12.5
Total	220,639	276,764	279,349	300,817	54,196	510,863	1,642,628	100.0
Percentage of State total	13.4	16.8	17.0	18.1	3.3	31.1	100.0	

[1] The distribution shown in this table is based on the analysis of a sample of 50,000 motor-vehicle registrations.

Registration fees.—Total license fees paid in 1930 amounted to $18,477,200. Passenger cars paid fees of $8, $12 or $20, depending on the horsepower, and motor trucks and busses paid $10, $12, $35, $75, or $250.

The total cost of the entire motor-vehicle department, including the licensing division, the investigators' division, and the chauffeurs' licensing division, was $667,036, which averaged 41 cents per motor vehicle registered. The actual cost of the automobile license department alone, without its allied investigators' and chauffeurs' divisions, was $482,160, or 29 cents per vehicle registered. The entire cost of the division was 3.6 percent of all the fees collected, and the cost of the automobile license department alone was 2.6 percent of all fees collected.

From the relationships disclosed by the sample of 50,000 vehicles, it was possible to locate motor-vehicle licenses paid by places, and the computations were found to give almost exactly the same totals by counties as were the actual amounts reported by the State authorities as having been paid in the same counties. Table 11 gives total registration fees paid and amounts

TABLE 11.—*Motor-vehicle registrations, persons per vehicle, and registration fees paid, distributed by place of ownership among the several classes of local government unit*

Place of ownership	Registration in 1930	Percentage of total	Persons per vehicle	Registration fees in 1930	Percentage of State total	Payments per vehicle
Townships:						
Automobiles	176,805	[1] 12.3	7.6	$1,731,400	[1] 12.5	$9.79
Trucks and busses	43,834	[2] 21.3	30.7	555,800	[2] 12.1	12.67
Total	220,639	13.4	5.9	2,287,200	12.4	10.37
Places up to 2,500:						
Automobiles	251,471	[1] 17.5	2.6	2,236,500	[1] 16.1	8.89
Trucks and busses	25,293	[2] 12.3	25.7	538,800	[2] 11.7	21.30
Total	276,764	16.8	2.4	2,775,300	15.0	10.03
Places 2,500 to 15,000:						
Automobiles	248,242	[1] 17.3	3.8	2,364,200	[1] 17.1	9.52
Trucks and busses	31,107	[2] 15.1	30.5	690,500	[2] 15.0	22.20
Total	279,349	17.0	3.4	3,054,700	16.6	10.94
Places 15,000 to 75,000:						
Automobiles	265,554	[1] 18.5	4.2	2,650,900	[1] 19.2	9.99
Trucks and busses	35,263	[2] 17.1	31.7	909,100	[2] 19.7	25.79
Total	300,817	18.4	3.7	3,560,000	19.3	11.83
Places 75,000 to 400,000:						
Automobiles	47,525	[1] 3.3	4.0	493,300	[1] 3.6	10.38
Trucks and busses	6,671	[2] 3.2	28.6	180,600	[2] 3.9	27.09
Total	54,196	3.3	3.5	673,900	3.7	12.43
Chicago:						
Automobiles	447,219	[1] 31.1	7.5	4,364,800	[1] 31.5	9.76
Trucks and busses	63,644	[2] 31.0	53.1	1,731,300	[2] 37.6	27.20
Total	510,863	31.1	6.6	6,096,100	33.0	11.93
State totals:						
Automobiles	1,436,816	87.5	5.3	13,841,100	75.0	9.63
Trucks and busses	205,812	12.5	37.1	4,606,100	25.0	22.38
Total	1,642,628	100.0	4.6	18,147,200	100.0	11.23

[1] Percentage of total for automobiles
[2] Percentage of total for trucks and busses.

per vehicle, distributed, by place of ownership, among the several classes of local government unit. The average fee paid for passenger automobiles was $9.63; for motor trucks and busses, $22.38; and the average for all motor vehicles registered was $11.23.

It will be observed that, while the average license fee paid for automobiles does not vary consistently, the fees paid for trucks and busses, as well as the averages for all motor vehicles, increase fairly steadily as the place of residence becomes more urban. It is also notable that trucks and busses, which constitute only 12.5 percent of the registrations, pay 25 percent of the license fees.

Mileage traveled and gasoline consumption.—Data on the average travel and gasoline consumption by Illinois trucks, busses, and automobiles in 1931 were obtained

through questionnaires sent to 20,000 motor vehicle owners in the State. As the questionnaire survey was conducted in 1932, it was impossible to expect motor vehicle owners to respond intelligently to requests for information covering the operation of their cars for any period except the year immediately preceding. For this reason the year 1931 was chosen, rather than 1930, which was the year covered by the financial survey. The sample was carefully selected to obtain a distribution in proportion to the motor-vehicle ownership of the various places, and every check indicates that a fair cross section was received. Usable replies were received from 7,150 owners.

The average mileage traveled and the average gasoline consumption by Illinois motor vehicles in 1931 are summarized in table 12. Ownership of the vehicles is classified according to place of residence of the owner. No attempt was made to obtain data as to the classification of the roads and streets on which this travel was performed. The facts summarized below were determined by questionnaire methods.

TABLE 12.—*Average miles traveled and average gasoline consumption in 1931, by place of ownership*

Place of ownership	Number of reports tabulated			Average gasoline consumption			Average miles traveled		
	Automobiles	Trucks and busses	All vehicles	Automobiles	Trucks and busses	All vehicles	Automobiles	Trucks and busses	All vehicles
				Gallons	*Gallons*	*Gallons*			
Townships	1,040	82	1,122	425	525	432	6,709	6,761	6,713
Places up to 2,500	1,030	92	1,122	567	997	603	8,651	10,154	8,775
Places 2,500 to 15,000	1,242	59	1,301	644	881	655	9,229	9,998	9,264
Places 15,000 to 75,000	1,278	85	1,363	672	987	691	9,174	9,204	9,176
Places 75,000 to 400,000	173	16	189	701	982	725	9,070	8,502	9,022
Chicago	1,881	172	2,053	666	1,500	736	8,546	9,285	8,608
State totals and averages	6,644	506	7,150	611	1,076	644	8,537	9,079	8,575

Further analysis of mileage and gasoline consumption, showing details of travel by groups of counties and by the two principal classes of motor vehicles, is given in table 13. The average passenger-car mileage for the State as a whole is shown to be 8,537 miles, with an average of 611 gallons of gasoline consumed.

Motor trucks and busses as a whole, in both urban and rural districts, traveled a slightly higher mileage than was the case with passenger automobiles. More significant is the fact that the average automobile consumed only 60 percent as much gasoline as was the average for trucks and busses. In the city of Chicago the gasoline consumption of trucks and busses was about two and one half times that of the average car.

No information was obtained as to the respective use of automobiles for business or pleasure purposes, or on the extent of travel outside of the State of Illinois.

From the facts ascertained by this study as to the location of motor vehicles, average miles traveled per vehicle, and the average gasoline consumption, it was possible to compute the total miles traveled in the State, and the total gasoline consumption. Some of these computations, which, of course, are only the most accurate approximations which can be made with the material available, are presented in table 14. Total mileage and gasoline consumption data are given for automobiles and for trucks and busses owned in each of the classes of local governmental unit.

TABLE 13.—*Motor-vehicle travel in 1931 by place of ownership in the 5 groups of counties* [1]

Item	County group					State totals and averages
	1	2	3	4	5	
Travel by automobiles:						
Average mileage traveled by automobiles owned in:						
Townships	7,733	7,621	6,447	6,052	6,052	6,709
Places up to 2,500	8,954	8,865	8,651	8,541	8,368	8,651
Places 2,500 to 15,000	9,414	9,171	8,961	9,597	8,761	9,229
Places 15,000 to 75,000	8,560	9,238	9,928			9,174
Places 75,000 to 400,000		9,070				9,070
Chicago	8,546					8,546
Total	8,632	8,858	8,493	7,986	7,180	8,537
Average gallons of gasoline consumed	663	634	560	510	476	611
Travel by motor trucks and busses:						
Average mileage traveled by motor trucks owned in:						
Townships	8,416	8,464	6,034	4,657	4,743	6,761
Places up to 2,500	7,257	7,531	13,907	9,066	12,096	10,154
Places 2,500 to 15,000	8,333	10,952	7,071	12,166	10,480	9,998
Places 15,000 to 75,000	5,264	10,089	9,970			9,204
Places 75,000 to 400,000		8,502				8,502
Chicago	9,285					9,285
Total	8,881	9,351	9,606	8,417	9,137	9,079
Average gallons of gasoline consumed	1,375	923	850	787	991	1,076
Travel by all motor vehicles:						
Average mileage traveled by motor vehicles owned in:						
Townships	7,794	7,704	6,422	5,969	5,972	6,713
Places up to 2,500	8,791	8,754	9,025	8,585	8,750	8,775
Places 2,500 to 15,000	9,402	9,290	8,884	9,737	8,989	9,264
Places 15,000 to 75,000	8,443	9,300	9,931			9,176
Places 75,000 to 400,000		9,022				9,022
Chicago	8,608					8,608
Total	8,649	8,896	8,562	8,016	7,344	8,575
Average gallons of gasoline consumed	712	657	578	529	519	644

[1] This table shows the variation in the mileage and gasoline consumption in the several groups of counties as indicated by the highway questionnaire sent out and tabulated during this survey.

TABLE 14.—*Mileage traveled, gasoline consumption, and gasoline-tax payments by automobiles and by trucks and busses, distributed by place of ownership*

Place of ownership	Registrations in 1930	Total travel	Percentage of total	Total gasoline consumption	Total gasoline tax paid	Percentage of total	Average per vehicle
		Millions of vehicle-miles		*Thousands of gallons*			
Townships:							
Automobiles	176,805	1,186.2		75,142	$1,926,400		$10.90
Trucks and busses	43,834	296.4		23,013	590,000		13.46
Total	220,639	1,482.6	10.5	98,155	2,516,400	8.9	11.41
Places up to 2,500:							
Automobiles	251,471	2,175.5		142,584	3,711,300		14.76
Trucks and busses	25,293	256.8		25,217	656,400		25.95
Total	276,764	2,432.3	17.2	167,801	4,367,700	15.5	15.78
Places 2,500 to 15,000:							
Automobiles	248,242	2,291.0		159,868	4,114,300		16.57
Trucks and busses	31,107	311.0		27,404	705,300		22.67
Total	279,349	2,602.0	18.3	187,272	4,819,600	17.1	17.25
Places 15,000 to 75,000:							
Automobiles	265,554	2,436.2		178,452	4,651,200		17.52
Trucks and busses	35,263	324.6		34,805	907,100		25.72
Total	300,817	2,760.8	19.5	213,257	5,558,300	19.6	18.48
Places 75,000 to 400,000:							
Automobiles	47,525	431.1		33,315	864,900		18.20
Trucks and busses	6,671	56.7		6,551	170,100		25.50
Total	54,196	487.8	3.4	39,866	1,035,000	3.7	19.10
Chicago:							
Automobiles	447,219	3,821.9		297,848	7,534,700		16.85
Trucks and busses	63,644	590.9		95,466	2,415,000		37.95
Total	510,863	4,412.8	31.1	393,314	9,949,700	35.2	19.48
State total:							
Automobiles	1,436,816	12,341.9	87.0	887,209	22,802,800	80.7	15.87
Trucks and busses	205,812	1,836.4	13.0	212,456	5,443,900	19.3	26.45
Total	1,642,628	14,178.3	100.0	1,099,665	28,246,700	100.0	17.20

The gasoline tax.—The total (net after refunds) gasoline tax paid (at the 1930 tax rate of 3 cents per gallon) was $28,246,700. The cost of collecting this tax and making the refunds was $79,805, or $2.83 per $1,000 collected.

TABLE 15.—*Average and total payments of motor vehicle fees and gasoline taxes by owners of motor vehicles in Illinois in 1930, distributed according to place of ownership*

Place of ownership	Payments per vehicle			Total payments					
	Registration fees	Gasoline tax	Total	Registration fees	Gasoline tax	Oil inspection fees	Wheel taxes	All motor vehicle imposts	Percentage of total
Townships	$10.37	$11.41	$21.78	$2,287,200	$2,516,400	$25,000		$4,828,600	8.9
Places up to 2,500	10.03	15.78	25.81	2,775,300	4,367,700	43,400	$68,700	7,255,100	13.4
Places 2,500 to 15,000	10.94	17.25	28.19	3,054,700	4,819,600	17,900	621,400	8,513,600	15.8
Places 15,000 to 75,000	11.83	18.48	30.31	3,560,000	5,558,300	55,200	726,800	9,900,300	18.3
Places 75,000 to 400,000	12.43	19.10	31.53	673,900	1,035,000	10,300	192,600	1,911,800	3.7
Chicago	11.93	19.48	31.41	6,096,100	9,949,700	98,900	5,565,000	21,709,700	40.1
State totals and averages	11.23	17.20	28.43	18,417,200	28,246,700	280,700	7,174,500	54,149,100	100.0

In addition to data on mileage and gasoline consumption table 14 also gives the total payments of gasoline tax, and the payments per vehicle. The amounts given as gasoline-tax payments do not equal the gallonage consumed multiplied by the tax of 3 cents per gallon. The reason for this apparent discrepancy is that the figures as to gallonage, average mileage, etc., are those obtained from the 1931 questionnaire; while the average tax paid is the actual net 1930 tax collected, apportioned on the basis of the facts as to allocation determined by this study. It is evident that there should be no great change between one year and the next in the proportionate amount of travel in the various groups of places, and the figures as given are, therefore, regarded as representative.

As shown in table 14, the average gasoline tax paid by automobiles was $15.87; that paid by trucks and busses was $26.45. The average for all motor vehicles was $17.20. The amount of gasoline tax paid per vehicle increases rather steadily as the place of residence becomes more urban, township vehicles paying only $11.41, while Chicago vehicles paid $19.48.

It will be observed that trucks and busses, which contributed only 13.0 percent of the traffic, paid 19.3 percent of the gasoline taxes.

The proceeds of the gasoline tax are used entirely for State and county highway programs. Two thirds of the tax is retained by the State and the balance returned to the counties. The county share of the gasoline tax may be used either on current highway programs or for the retirement of bonds. The payment of the allotments to counties is under the jurisdiction of the Illinois Division of Highways. Counties cannot receive their allotments until complete plans for the disposition of these funds have been submitted to the Division of Highways and approved by it.

TOTAL MOTOR-VEHICLE PAYMENTS SHOWN

The distribution, according to place of ownership, of all registration fees and gasoline taxes paid by owners of motor vehicles is shown in table 15. Both total and per-vehicle figures are given. In addition to the license fees and the gasoline tax, the State imposes an "oil-inspection fee" at the rate of 1 cent per 100 gallons; and various municipalities impose privilege taxes, which are known as "wheel taxes", upon motor vehicles. These payments are also shown; so that table 15 gives the total af all imposts upon motor vehicles and their use in Illinois in 1930.

DATA ON MOTOR-VEHICLE TAXES AND TRAVEL SUMMARIZED

Tables 16 and 17 summarize the relationships between population, vehicles registered, registration fees and gasoline taxes paid, and traffic, which have been brought out in the preceding paragraphs. Table 16 shows comparisons between the various classes of local government unit; in table 17 the groups of counties are compared.

TABLE 16.—*Percentage comparison of the several classes of local government unit as to population, motor vehicles registered, motor-vehicle taxes paid, and share of total State traffic contributed by vehicles owned in these places*

Unit of Government	Population	Motor vehicles registered	Registration fees paid	Gasoline taxes paid	Registration fees and gasoline taxes paid	All motor vehicle imposts [1] paid	Contribution to total traffic
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Townships	17.61	13.4	12.4	8.9	10.3	8.9	10.5
Places up to 2,500	8.53	16.8	15.0	15.5	15.3	13.4	17.2
Places 2,500 to 15,000	12.45	17.0	16.6	17.1	16.9	15.8	18.3
Places 15,000 to 75,000	14.66	18.4	19.3	19.6	19.5	18.3	19.5
Places 75,000 to 400,000	2.50	3.3	3.7	3.7	3.7	3.5	3.4
Chicago	44.25	31.1	33.0	35.2	34.3	40.1	31.1
Total	100.00	100.0	100.0	100.0	100.0	100.0	100.0

[1] Includes oil-inspection fees and wheel taxes.

TABLE 17.—*Percentage comparison of the several groups of counties as to population, motor vehicles registered, and motor-vehicle taxes paid.*

County group	Population	Motor vehicles registered	Registration fees paid	Gasoline taxes paid	Registration fees and gasoline taxes paid
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
1	52.19	39.4	41.9	43.2	42.7
2	21.74	24.3	25.6	25.8	25.7
3	14.30	19.4	17.9	17.3	17.5
4	9.39	13.8	12.1	11.2	11.6
5	2.38	3.1	2.5	2.5	2.5
Total	100.00	100.0	100.0	100.0	100.0

From these tables and those previously given the following conclusions have been derived:

1. The townships, with 17.61 percent of the population, contained 13.4 percent of the registered motor vehicles; motor-vehicle owners in the townships paid 10.3 percent of the motor-vehicle registration fees and gasoline taxes and contributed 10.5 percent of the total traffic of Illinois vehicles.

2. Incorporated places other than Chicago, with 38.14 percent of the population, contained 55.5 percent of the registered motor vehicles; motor-vehicle owners in these places paid 55.4 percent of the motor-vehicle registration fees and gasoline taxes, and contributed 58.4 percent of the total traffic of Illinois vehicles.

3. Chicago, with 44.25 percent of the population, contained 31.1 percent of the registered motor vehicles;

motor-vehicle owners in Chicago paid 34.3 percent of the motor-vehicle registration fees and gasoline taxes and contributed 31.1 percent of the traffic of Illinois vehicles.

4. Of all imposts on motor vehicles, the owners of motor vehicles in the townships paid 8.9 percent; those in incorporated places other than Chicago paid 51.0 percent; and those in Chicago paid 40.1 percent.

5. The average registration fee paid was $11.23; the average gasoline tax payment, $17.20; the total, $28.43. Both registration fees and gasoline tax payments were found to increase as the place of residence becomes more urban.

6. The average registration fee paid by passenger automobiles was $9.63; by trucks and busses, $22.38. The average gasoline-tax payments by passenger automobiles were $15.87; by trucks and busses, $26.45. Thus the average payments in registration fees and gasoline taxes were: passenger automobiles, $25.50; trucks and busses, $48.83.

7. The owners of motor trucks and busses, which constituted 12.5 percent of the total registrations and contributed 13 percent of the total traffic of Illinois vehicles, paid 25 percent of the registration fees and 19.3 percent of the gasoline taxes.

SPECIAL ASSESSMENTS AND THEIR PLACE IN THE HIGHWAY PROGRAM

In Illinois as in many other States, in cities and in municipal communities, streets and other public improvements are largely financed through special assessments. In Illinois cities in 1930, special assessments collected for highway purposes totaled $56,260,200. At least $50,000,000 of special-assessment highway-improvement bonds were issued. Practically all the costs of paving streets in cities of 10,000 and over, and also in many places smaller than this, are met through special assessments.

The special assessment obligations, not being a part of the cities' financial affiairs, are not treated as a part of urban debt, and hence, are not subject to the salutary effects of debt limitation statutes. This makes possible such situations as exist, for example, in Niles Center, an Illinois city of 5,007 persons, having a special assessment debt outstanding of $10,208,900, or over $2,000 for every man, woman, and child in the city. Or, as an extreme case, the village of Westchester, a subdivision in Cook County, with a population of 358 persons, has an assessed value of $2,369,200 and special-assessment obligations outstanding of $3,375,577, or nearly $10,000 per capita. While this is exceptional, there are many Illinois cities which have special-assessment obligations outstanding of from 10 to 50 percent of their assessed valuation. For example, Lombard, in Du Page County, with an assessed valuation of $4,812,900, had outstanding special assessment obligations of approximately $2,750,000, or about $450 per capita. Villa Park, in Du Page County, with a population of 6,220 and an assessed valuation of $4,533,900, had outstanding special assessment obligations of about $4,000,000.

So far as is possible, these expenditures and receipts and debts have been assembled in this study. The highway expenditures include all of the public payments in Illinois for roads and streets, including those financed through special assessments.

With the exception of the Illinois Central charter line system, railroads pay upon the property allocated by the tax commission to the community the same taxes as are charged against other property in the community. This caused them to contribute in 1930 $3,658,400 toward the local highway program.

Many Illinois cities charge wheel taxes, usually from $3 to $5 per vehicle, for the use of streets. Occasionally revenues are derived from franchises for the use of certain streets by utilities. Illinois urban communities derived $7,174,500 from these sources within the year, of which $5,565,000 was in the city of Chicago.

While motor vehicles in Illinois are subject to the personal-property tax, the lax administration of the law makes these taxes more of a theory than a fact. Probably more cars are omitted than are assessed, and those which are listed usually are valued at nominal amounts. No reasonably accurate figures as to this tax could be obtained without an unwarranted expenditure.

BONDED INDEBTEDNESS

All units of Government in Illinois may bond. At the end of 1930 the total outstanding funded debt of the State and its subdivisions was $959,238,400, and of this amount $424,722,300, or 44.3 percent, was incurred for streets and highways. The remainder was distributed as follows: 8.6 percent for education, 43.5 percent for the protection of persons and property, and 3.6 percent for Government. In addition to the funded debt mentioned above, there were temporary borrowings outstanding of the city of Chicago of $259,013,000, which in all probablity will be eventually funded.

The State indebtedness stood at $200,462,000, or a little over one fifth of the total public funded indebtedness, and of this sum $148,010,000, or 73.8 percent, was incurred for the financing of highways. This is a self-liquidating debt now in the process of retirement, certain specific motor-vehicle receipts being set aside for this purpose.

It may be explained that the purpose in creating a State highway system was to construct as rapidly as possible a 10,000-mile system of highly developed roads which would be planned to serve the transportation needs of the State and would be developed and maintained in an intelligent manner. The entire project was to be controlled by the State, and, with the exception of Federal-aid appropriations, financed by the State.

At the outset this system was financed through bond issues, the first issue being one of $60,000,000 authorized in 1918, and a second of $100,000,000 authorized in 1924. Of the total bond issues authorized of $160,000,000, $158,000,000 were sold. These bonds are now being retired. Forty-eight hundred miles of the State primary system were provided for by the first bond issue of $60,000,000, and 5,000 miles additional by the $100,000,000 program. The last of the bond-issue money, amounting to $2,047,000, was expended in 1930.

The county indebtedness was comparatively small, consisting of $49,990,000 for Cook County, and $10,498,900 for all of the other counties in the State, only 20 of the 102 counties having any bonded indebtedness whatever. Of this county indebtedness, the major portion is for the highway program.

Municipalities had a total indebtedness of $673,157,100, by far the greater portion of this being in the form of special assessment bonds.

Townships had an indebtedness of $25,130,400 which, outside of school-district obligations, was practically

entirely for township roads and was distributed widely throughout the State.

The total bonds outstanding on December 31, 1930 ($959,238,400) were 12.4 percent of the assessed valuation of the State, and the per capita indebtedness was $125.71. The per capita cost of all debt service in 1930 was $15.42.

DEBT SERVICE

Debt service consists of the interest and principal payments on indebtedness. In Illinois in 1930 the total debt-service payments were $117,586,400, of which $53,929,100, or 45.9 percent, were for highways; $8,308,600, or 7.1 percent, for education; $53,318,700, or 45.3 percent, for public benefit; and $2,030,000, or 1.7 percent, for government.

With the exception of State payments, as a general rule about two thirds of the debt service consisted of interest payments and the balance, principal. Details are shown in table 18.

While per capita debt figures are given, in considering such figures caution should be observed. Wide variance in local conditions prevents a general average from having any great significance. It is possible that the debt presentation is not entirely complete. The major portion of the municipal indebtedness is in the form of special-assessment obligations, the records as to which are in extremely poor condition and difficult of access. The same condition is found with respect to the township obligations. Every reasonable effort has been used to obtain complete information as to indebtedness. It is believed that the percentage of the total which may have been omitted is extremely small.

TABLE 18.—*Payments for debt service in 1930*

Unit of government	Principal	Interest	Total
State	$81,113,000	$8,401,300	$12,814,300
Per capita	0.58	1.10	1.68
Cook County	56,099,800	24,764,800	80,864,600
Per capita	14.09	6.22	20.31
All other units	16,209,300	7,698,200	23,907,500
Per capita	4.44	2.11	6.55
Total	76,722,100	40,864,300	117,586,400
Per capita	10.06	5.36	15.42

delinquent. Furthermore, tax levies and expenditures can never agree because of a difference in the time element. Taxes are levied to meet anticipated expenditures and also to defray costs which have been incurred in the past and defrayed through revenues received from the sale of securities and other borrowings. Expenditures are made out of funds received from current taxes, from surpluses, from balances on hand, and from borrowings.

Where old indebtedness is being retired and comparatively few new obligations are incurred, taxes will normally exceed expenditures. Where current work is to a considerable extent financed through borrowings, then expenditures will exceed taxes.

Table 19 itemizes the actual expenditures by units of Government with a per capita distribution of expenditures for the four major governmental purposes.

The various governmental agencies of the State perform various functions of public service. The State assumes the support of the through State highway system. This item and public benefit are major items of State expenditures. The public benefits administered by the State are correctional and public health institutions for the treatment of persons from all parts of the State; relief of catastrophes, such as droughts, floods, and wars; and State regulatory and conservation bodies. The educational activities of the State

ILLINOIS EXPENDITURES IN 1930

The total expenditures for all purposes (exclusive of principal payments on bonds and loans) by all units of Government in Illinois in 1930 were $551,249,500, of which $166,298,900 was expended for highways and streets, $167,599,300 for education, $188,307,900 for public benefit, and $29,043,400 for other governmental purposes.

Expenditures by the State were $91,079,500; by the counties, $56,310,600; by Chicago, $245,989,600; by other incorporated places, $120,479,200; by the townships, $37,390,600.

Of the total expenditures, 19.4 percent was made in rural areas (townships); 5.9 percent in incorporated places having a population up to 2,500; 9.9 percent in places of 2,500 to 15,000 population; 10.8 percent in places of 15,000 to 75,000 population; 1.8 percent in places of 75,000 to 400,000 population; and 52.2 percent in Chicago.

GENERAL EXPENDITURES ANALYZED

The total of all public charges levied against persons and property in Illinois for collection in the year 1930 was $582,407,100, or $76.32 per capita. The total of all expenditures for the same year was $551,249,500, of which amount $166,298,900, or 30.1 percent, was expended for highways; $167,599,300 or 30.4 percent, for education; $188,307,900, or 34.2 percent, for public benefit and protection; and $29.043,400, or 5.3 percent, for government.

These expenditures include $40,864,300 in interest payments on funded indebtedness, as well as interest on the short-term borrowings of municipalities. The principal payments of $76,722,100 are not included, as they represent repayments of funds charged as expenditures in previous years.

There can be no exact agreement between receipts and expenditures. In the first place the amounts shown as taxes are the amounts levied. Eventually, practically all of these sums undoubtedly will be collected. For any one particular calendar year, however, it is possible that comparatively large sums may remain

TABLE 19.—*Total expenditures in 1930 by the State, the counties, and the local units of government; and per capita expenditures by purpose*

Expended by—	Amount	Percent	Per capita				
			Highways and streets	Education	Public benefit	Government	Total
State	$91,079,500	16.5	$5.30	$1.32	$4.80	$0.52	$11.94
Counties	56,310,600	10.2	2.21	.02	3.79	1.36	7.38
Townships	37,390,600	6.8	6.65	19.29	1.03	.86	27.83
Places up to 2,500	23,404,300	4.2	7.13	17.89	8.99	1.93	35.94
Places 2,500 to 15,000	42,727,400	7.8	9.16	21.35	12.75	1.73	44.99
Places 15,000 to 75,000	46,701,000	8.5	8.02	20.56	11.42	1.75	41.75
Places 75,000 to 400,000	7,646,500	1.4	10.05	16.03	13.09	.90	40.07
Chicago	245,989,600	44.6	22.47	21.75	26.10	2.53	72.85
Total	551,249,500	100.0	21.79	21.96	24.68	3.81	72.24

TABLE 20.—*Percentage relationships of expenditures by units of Government and by purpose*

Expended by—	Percentages of expenditures of the units of Government for the purposes indicated					Percentages of the total expenditure for each purpose made by each unit of Government				
	Highways and streets	Education	Public benefit	Government	All purposes	Highways and streets	Education	Public benefit	Government	All purposes
State	44.4	11.1	40.2	4.3	100.0	24.3	6.0	19.5	13.6	16.5
Counties	29.9	.3	51.4	18.4	100.0	10.1	.1	15.4	35.7	10.2
Townships	23.9	69.3	3.7	3.1	100.0	5.4	15.5	.7	4.0	6.8
Places up to 2,500	19.8	49.8	25.0	5.4	100.0	2.8	7.0	3.1	4.3	4.2
Places 2,500 to 15,000	20.4	47.5	28.3	3.8	100.0	5.2	12.1	6.4	5.7	7.8
Places 15,000 to 75,000	19.2	49.2	27.4	4.2	100.0	5.4	13.7	6.8	6.7	8.5
Places 75,000 to 400,000	25.1	40.0	32.7	2.2	100.0	1.2	1.8	1.3	.6	1.4
Chicago	30.8	29.9	35.8	3.5	100.0	45.6	43.8	46.8	29.4	44.6
Total	30.1	30.4	34.2	5.3	100.0	100.0	100.0	100.0	100.0	100.0

are primarily the conduct of institutions of higher learning. Under the heading "Government" are included the primary executive and administrative charges of the various units.

A general distribution of expenditures under the four heads and by various units of government, expressed in percentages, is conveniently shown in table 20.

The distribution of expenditures according to the classes of local unit in which they were made is shown, in per capita form, in table 21.

TABLE 21.—*Per capita expenditures by purpose and by classes of local unit in which the expenditures were made*

Expended in—	Streets and highways	Education	Public benefit	Government	Total
Townships	[1] $46.72	$20.25	$9.53	$2.70	[1] $79.20
Places up to 2,500	9.66	19.07	17.57	3.65	49.95
Places 2,500 to 15,000	10.43	22.48	21.13	3.58	57.62
Places 15,000 to 75,000	8.26	21.70	19.82	3.59	53.37
Places 75,000 to 400,000	10.06	17.47	21.78	2.79	52.10
Chicago	[1] 22.56	23.08	35.07	4.57	[1] 85.28
Total	21.79	21.96	24.68	3.81	72.24

[1] These figures differ slightly from corresponding values given in the report published by the State of Illinois. For explanation, see footnote 5.

TABLES SHOW RELATIONS BETWEEN VALUATION, TAXATION, EXPENDITURES, AND OTHER FACTORS

The study disclosed interesting relationships among the factors of taxation, expenditures, valuation, population, etc. The valuation, taxation, and expenditures per capita in 1930 are shown in table 22.

TABLE 22.—*Per capita valuation taxation, and expenditures in 1930, in the various classes of local unit and in the State as a whole*

Unit of government	Valuation	Taxation	Expenditures	Ratio of per capita expenditures to per capita taxation
Townships	$1,494	$53.22	[1] $79.20	[1] 1.48
Places up to 2,500	712	58.37	49.95	.86
Places 2,500 to 15,000	713	69.05	57.62	.83
Places 15,000 to 75,000	778	65.92	53.37	.81
Places 75,000 to 400,000	906	63.24	52.10	.82
Chicago	1,045	95.21	[1] 85.28	[1] .90
Entire State	1,012	76.32	72.24	.95

[1] These figures differ slightly from corresponding values given in the report published by the State of Illinois. For explanation, see footnote 5.

The ratio of per capita expenditures to per capita taxation is a rather accurate index of the flow of State (including Federal) and county aids to the less populous areas, except as affected by abnormal borrowings. For instance, in the townships per capita expenditures exceeded per capita taxation by $25.98, or about 33 percent. Expenditures in municipalities, on the other hand, were less than taxes and this difference would be even greater (particularly in Chicago) if borrowings were also considered as a source of revenue. Table 22 indicates in general this shift in funds.

Table 23 gives percentage relationships between population, motor-vehicle ownership, valuation, taxes payable, and expenditures made in the several classes of local unit in 1930.

TABLE 23.—*Percentage distribution of population, motor-vehicle ownership, property valuation, taxes paid, and expenditures made in the several classes of local unit in 1930*

Unit of government	Population	Motor-vehicle ownership	Valuation	Taxes paid	Expenditures made
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Townships	17.6	13.4	26.0	12.3	19.4
Places up to 2,500	8.5	16.8	6.0	6.5	5.9
Places 2,500 to 15,000	12.4	17.0	8.8	11.2	9.9
Places 15,000 to 75,000	14.7	18.4	11.3	12.7	10.8
Places 75,000 to 400,000	2.5	3.3	2.2	2.1	1.8
Chicago	44.3	31.1	45.7	55.2	52.2
Total	100.0	100.0	100.0	100.0	100.0

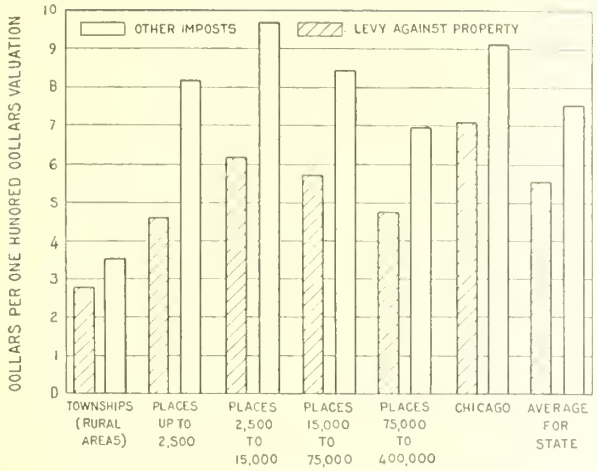
HIGHWAY EXPENDITURES DISCUSSED IN DETAIL

There was expended on all highways and streets in Illinois in 1930 a total of $166,298,900, which exceeded the total taxes imposed for highways by $16,032,200. The total expenditures were divided as follows: On the State highway system, $40,421,100, or 24.3 percent; on the county systems, $16,840,100, or 10.1 percent; on the local roads, $8,937,500, or 5.4 percent. Street expenditures by municipal authorities were as follows: On Chicago streets, $75,873,300, or 45.6 percent of the State total; and on other streets, $24,226,900, or 14.6 percent.

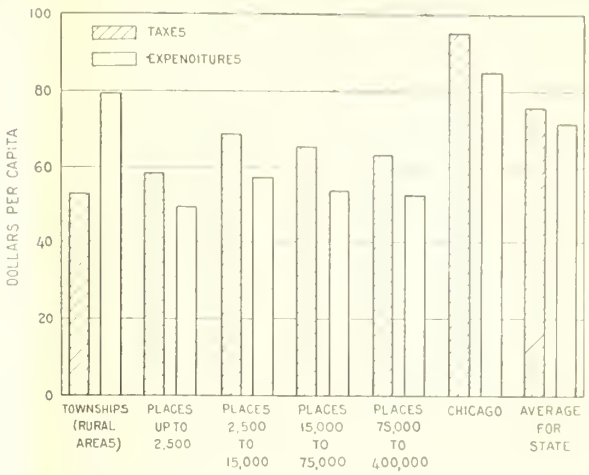
The sum of $3,155,000 was spent by the Division of Highways on parts of the State trunk system lying within the limits of incorporated places; of this amount $22,500 was spent in Chicago. In addition, Cook County expended $265,400 on streets in Chicago which form a part of the county trunk system.[5]

Adding these amounts to the amounts expended by the municipalities themselves, we find that the total spent on urban streets was $103,520,600; the total expenditures on rural roads being $62,778,300.

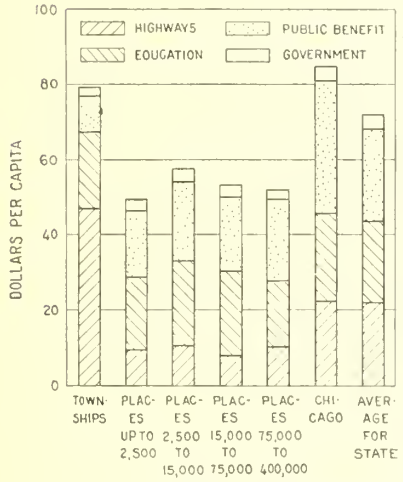
[5] At the time of preparation of the "Master statistical tables", which form a part of the full report published by the State of Illinois, the expenditures of Cook County in Chicago could not be ascertained. For this reason there are slight differences, negligible in effect, between figures given in tables 21, 22, and 25 and corresponding figures in the full report.



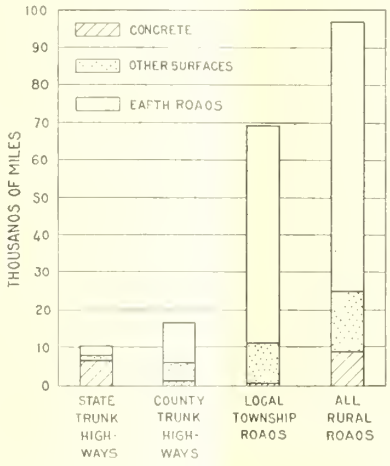
AVERAGE TAX RATES ON GENERAL PROPERTY IN THE SEVERAL CLASSES OF LOCAL UNIT AND THE STATE AS A WHOLE, AND RATES REQUIRED IF ALL TAXES WERE LEVIED ON GENERAL PROPERTY (TABLE 6)



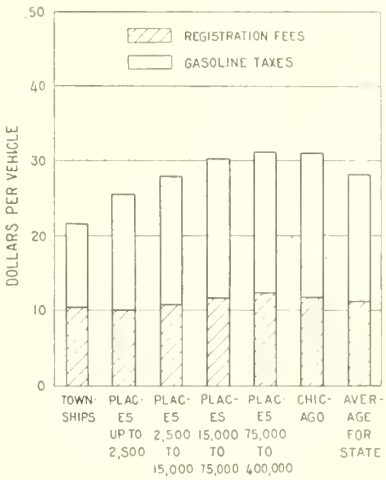
PER CAPITA TAXES AND PER CAPITA EXPENDITURES IN THE SEVERAL CLASSES OF LOCAL UNIT AND THE STATE AS A WHOLE (TABLE 22)



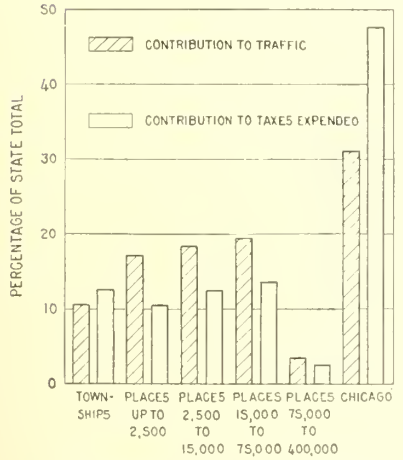
PER CAPITA EXPENDITURES BY PURPOSE IN THE SEVERAL CLASSES OF LOCAL UNIT AND THE STATE AS A WHOLE (TABLE 21)



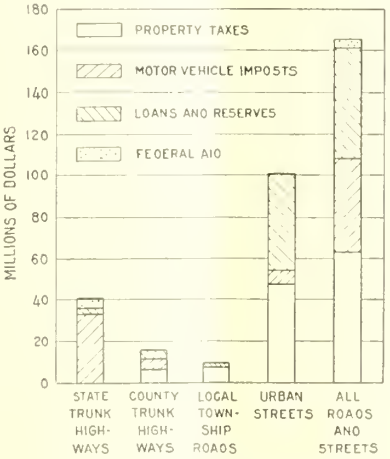
TOTAL MILEAGE OF RURAL HIGHWAY SYSTEMS AND SURFACED MILEAGE BY TYPES (TABLE 1)



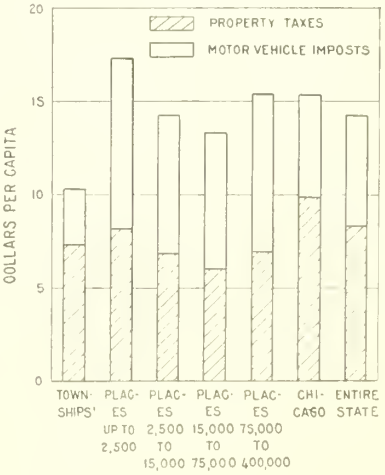
AVERAGE REGISTRATION FEES AND GASOLINE TAX PAYMENTS BY MOTOR VEHICLE OWNERS IN THE SEVERAL CLASSES OF LOCAL UNIT AND IN THE STATE AS A WHOLE (TABLE 15)



COMPARISON BETWEEN THE CONTRIBUTIONS BY MOTOR VEHICLES DOMICILED IN THE SEVERAL CLASSES OF LOCAL UNIT TO THE TOTAL TRAVEL OF ILLINOIS VEHICLES, AND THE CONTRIBUTIONS BY TAXPAYERS IN THESE SAME UNITS TO CURRENT TAXES EXPENDED ON ROADS AND STREETS (TABLES 14 AND 27)



TOTAL EXPENDITURES ON HIGHWAY SYSTEMS AND URBAN STREETS SHOWING SOURCES OF FUNDS (TABLE 27)



PER CAPITA CONTRIBUTIONS BY RESIDENTS OF THE SEVERAL CLASSES OF LOCAL UNIT TO CURRENT TAXES EXPENDED ON ROADS AND STREETS (BASED ON TABLE 27)

GRAPHICAL SUMMARY OF ESSENTIAL FACTS DISCLOSED BY SURVEY OF ILLINOIS FINANCES IN 1930

All expenditures on the State highway systems were made under the supervision of the Division of Highways, and the total of these expenditures was $40,421,100. A survey [6] conducted by the State of Illinois indicates that the total annual traffic is in the neighborhood of 3,419,000,000 vehicle-miles. The expenditures on the State system were, therefore, approximately 1.18 cents per vehicle-mile.

The total expenditures for construction and maintenance on the State system were $31,954,300. The remaining expenditures, representing general undistributed overhead of the highway department and the interest on State highway bonds, totaled $8,466,800. Table 24 gives the distribution of construction and maintenance expenditures among the subdivisions of the State system.

Table 25 gives the amounts expended for streets and highways in the several classes of local unit. Of the $62,778,300 expended in the townships (i.e., on rural roads), $37,266,100 was expended by the Division of Highways on the State system; $16,574,700 by the counties on the county trunk system; and $8,937,500 by the local authorities on the local (township) roads.

EXPENDITURES AND TAXES FOR STREETS AND HIGHWAYS COMPARED

Table 25 also gives, for purposes of comparison, the street and highway taxes paid (or payable) by taxpayers and motor-vehicle owners in the several classes of local unit. These figures were previously given in table 8. The last column of table 25 gives the ratio of expenditures to taxes.

There was spent upon the rural highway system, $62,778,300. The rural units of government contributed in all forms of highway taxes, $16,996,900. In other words, the highway expenditures in the townships were 369 percent of the revenues derived from these units of government. This shows a decided flow of revenues from the villages and cities toward the rural areas. The conclusion should not be hastily drawn that this represents an unfair subsidizing of the rural districts at the expense of the urban communities. It is well known that a large portion of the traveling done by urban vehicles is in the rural areas, particularly on the State highways. Furthermore, the transportation facilities provided by good rural roads have decided influences upon urban commercial enterprises. The benefits received by the cities as a result of rural highway development tend to counterbalance the flow of revenues.

As a general rule, the less dense the population of the county, the greater its percentage of gain in the comparison of expenditures with taxation. While the townships in each county invariably received more in expenditures than they paid in taxes, the reverse was true for practically every group of incorporated places.

To avoid incorrect conclusions, such facts must also be carefully examined. For example, in the case of Chicago, it can be seen that the city paid $70,913,600 in highway taxes and that the expenditures were $76,161,200. It would seem, therefore, that the city of Chicago received more than it paid, but such is far from the case. During the year, Chicago incurred temporary and long-term indebtedness for highway purposes of $41,491,000. Actually, Chicago contributed about $18,995,400 to the State and county highway programs and there was expended by these agencies within the city the total amount of $287,900.

It is also incorrect to assume that because Chicago contributed this $18,995,400 to the State and county and received but $287,900 in the form of expenditures, the difference necessarily will be spent in other communities. Various State and county highway funds are created from the highway receipts. It is easily possible that a community may in one year make a contribution to a fund and in the next year receive a benefit from that fund in the form of an expenditure. This is especially true of payments made in the later months of the calendar year.

In considering revenues as compared with expenditures, it is also important to remember that there is a natural lag between the revenues and the expenditures.

SOURCES OF REVENUES FROM WHICH FUNDS WERE DERIVED FOR 1930 HIGHWAY EXPENDITURES

The first classification of the source of funds is by the governmental agency whose act made the funds available. These funds so classified are presented in table 26.

RELATIONS BETWEEN HIGHWAY TAXES PAID, HIGHWAY EXPENDITURES, AND TRAVEL

The total mileage of rural highways in Illinois in 1930 was 97,234. The State highway system consisted of 10,098 miles, divided into 6,976 miles of Federal-aid highways and 3,122 miles of other State trunk highways. The county highway system contained 17,369 miles, and the local township roads, 69,767 miles. No data are available on mileage of village and city streets.

The total volume of traffic (exclusive of out-of-State vehicles) on all roads and streets in Illinois during the year 1930 was approximately 14 billion vehicle-miles, of which 10.5 percent was produced by vehicles of rural ownership; 17.2 percent by vehicles owned in incorporated places having a population up to 2,500; 18.3 per cent by vehicles owned in places of 2,500 to 15,000 population; 19.5 percent by vehicles owned in places of 15,000 to 75,000 population; 3.4 percent by vehicles owned in places of 75,000 to 400,000 population; and 31.1 percent by vehicles owned in Chicago.

Expenditures on the State highway system in 1930 were $40,421,100; on the county systems, $16,840,100; on the local (township) roads, $8,937,500; on Chicago streets, $75,873,300; and on other streets, $24,226,900.

Of the total property taxes expended on all roads and streets, 10.9 percent was expended on county trunk highways; 13.8 percent on the local (township) roads; and 75.3 percent on urban streets.

Of the total motor vehicle taxes expended on all roads and streets, 72.8 percent was expended on State trunk highways; 11.5 percent on county trunk highways, and 15.7 percent on urban streets.

Of the total of all taxes and imposts expended on all roads and streets, rural property and motor vehicle owners paid 12.8 percent, and travel by rural vehicles made up 10.5 percent of the total travel on all roads and streets; city and village property and motor vehicle owners paid 87.2 percent, and travel by city and village vehicles made up 89.5 percent of the total travel.

[6] Report of the Survey of Traffic on Illinois State Highways, August 1931 to Labor Day, 1932, State of Illinois, 1932, p. 19.

TABLE 24.—*Expenditures for construction and maintenance on the State highway system in 1930*

Highway system	Construction expenditures		Maintenance expenditures		Total		Percentage of total construction and maintenance
	Amount	Percent	Amount	Percent	Amount	Percent	
Federal-aid (primary)	$6,275,000	80.9	$1,478,200	19.1	$7,753,200	100.0	24.3
Federal-aid (secondary)	12,913,300	91.1	1,261,400	8.9	14,174,700	100.0	44.3
Other State highways	9,428,700	94.0	597,700	6.0	10,026,400	100.0	31.4
Total	28,617,000	89.6	3,337,300	10.4	31,954,300	100.0	100.0

TABLE 25.—*Street and highway expenditures made in the several classes of local unit in 1930, and comparison of expenditures with charges imposed for streets and highways upon taxpayers and motor-vehicle owners in these classes of unit*

Class of local unit	Street and highway expenditures		Street and highway taxes [1]	Ratio of expenditures to taxes
	Amount	Percent		
Townships	$62,778,300	37.7	$16,996,900	3.69
Places up to 2,500	6,294,400	3.8	15,059,300	.42
Places 2,500 to 15,000	9,907,200	6.0	21,479,300	.46
Places 15,000 to 75,000	9,237,300	5.5	22,114,800	.42
Places 75,000 to 406,000	1,920,500	1.2	3,702,800	.52
Chicago	76,161,200	45.8	70,913,600	1.07
Total	$166,298,900	100.0	$150,266,700	1.11

[1] See table 8.

TABLE 26.—*Funds expended on the several highway systems and the urban streets in 1930, and the approximate amounts and percentages of these funds provided by imposts made by the various governmental units, and by loan and reserve funds*

Highway system and form of revenue	Governmental agency providing funds				Total	Percentage of total current tax funds	Percentage of total funds
	Federal government	State	Counties	Local governments			
State trunk highways:							
Federal-aid system:							
Loans or reserves		$2,047,000			$2,047,000		
Current taxes	$4,089,900	$21,592,000			$25,681,900	22.8	
Total	$4,089,900	$23,639,000			$27,728,900		16.7
Percent	14.7	85.3			100.0		
State bond-issue system:							
Loans or reserves		$1,097,400			$1,097,400		
Current taxes		$11,594,800			$11,594,800	10.3	
Total		$12,692,200			$12,692,200		7.6
Percent		100.0			100.0		
Entire State system:							
Loans or reserves		$3,144,400			$3,144,400		
Current taxes	$4,089,900	$33,186,800			$37,276,700	33.1	
Total	$4,089,900	$36,331,200			$40,421,100		24.3
Percent	10.1	89.9			100.0		
County highways (State-aid):							
Loans or reserves		$1,267,900	$3,451,900		$4,719,800		
Current taxes		$5,256,500	$6,863,800		$12,120,300	10.8	
Total		$6,524,400	$10,315,700		$16,840,100		10.1
Percent		38.7	61.3		100.0		
Local rural (township) roads:							
Loans or reserves				$265,000	$265,000		
Current taxes				$8,672,500	$8,672,500	7.7	
Total				$8,937,500	$8,937,500		5.4
Percent				100.0	100.0		
Urban streets outside Chicago: [1]							
Loans or reserves				$6,369,500	$6,369,500		
Current taxes				$17,857,400	$17,857,400	15.8	
Total				$24,226,900	$24,226,900		14.6
Percent				100.0	100.0		
Chicago streets: [2]							
Loans or reserves				$39,104,500	$39,104,500		
Current taxes				$36,768,800	$36,768,800	32.6	
Total				$75,873,300	$75,873,300		45.6
Percent				100.0	100.0		
All roads and streets:							
Loans or reserves		$4,412,300	$3,451,900	$45,739,000	$53,603,200		
Current taxes	$4,089,900	$38,443,300	$6,863,800	$63,298,700	$112,695,700	100.0	
Total	$4,089,900	$42,855,600	$10,315,700	$109,037,700	$166,298,900		100.0
Percent	2.5	25.8	6.2	65.6	100.0		

[1] Does not include expenditures by Division of Highways on streets which are part of the State trunk system.
[2] Does not include expenditures by Division of Highways and Cook County on streets which are parts of the State and county systems.

It will be seen from the table that of all the revenues, the Federal Government provided 2.5 percent and the State 25.8 percent. The State highway system, consisting of the Federal-aid highways and the State-bond-issue system, is supported jointly by funds provided by the United States and the State. The county highway system is a joint enterprise between the State and the counties, about two thirds of the funds being provided through the acts of county authorities. The funds for rural roads and the city streets, except for those included as part of the other highway systems, are all provided through acts of the local officials.

Of the total expenditures for highways and streets, $4,089,900 was Federal-aid money, $53,603,200 was derived from loans and reserves, and $108,605,800 was spent out of current receipts from property taxation and motor-vehicle imposts. Table 27 gives the breakdown of this latter sum into payments by taxpayers and motor-vehicle owners in the several classes of local unit; and thus shows the contributions by each of these classes to the expenditures in 1930 on the several highway systems and the local streets. The Federal-aid money is naturally not subject to such an analysis; and the loans and reserves do not represent taxes paid in

TABLE 27.—*Approximate amounts of the 1930 taxes and imposts expended on the current highway program, listed according to highway system, type of tax, and class of local unit in which the tax was paid*

IMPOSTS ON GENERAL PROPERTY

Paid by taxpayers in—	Highway system										
	State trunk		County trunk		Township local		Urban streets		All highways and streets		
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Percentage of total
Townships			$3,051,700	30.9	$6,831,700	69.1			$9,883,400	100.0	15.7
Places up to 2,500			383,900	7.1	739,700	13.8	$4,257,800	79.1	5,381,400	100.0	8.5
Places 2,500 to 15,000			710,700	11.0	833,000	12.9	4,922,000	76.1	6,465,700	100.0	10.3
Places 15,000 to 75,000			578,000	8.6	268,100	4.0	5,872,300	87.4	6,718,400	100.0	10.7
Places 75,000 to 400,000			132,200	10.0			1,195,800	90.0	1,328,000	100.0	2.1
Chicago			2,007,300	6.0			31,203,800	94.0	33,211,100	100.0	52.7
Total			6,863,800	10.9	8,672,500	13.8	47,451,700	75.3	62,988,000	100.0	100.0

IMPOSTS ON MOTOR VEHICLES

Townships	$3,571,200	88.4	$467,800	11.6					$4,039,000	100.0	8.9
Places up to 2,500	5,071,400	85.2	814,700	13.7			$68,700	1.1	5,954,800	100.0	13.1
Places 2,500 to 15,000	5,565,900	78.5	898,900	12.7			621,400	8.8	7,086,200	100.0	15.5
Places 15,000 to 75,000	6,468,200	78.7	1,030,300	12.5			726,800	8.8	8,225,300	100.0	18.0
Places 75,000 to 400,000	1,228,000	76.0	194,500	12.1			192,600	11.9	1,615,100	100.0	3.5
Chicago	11,282,100	60.3	1,850,300	9.9			5,565,000	29.8	18,697,400	100.0	41.0
Total	33,186,800	72.8	5,256,500	11.5			7,174,500	15.7	45,617,800	100.0	100.0

ALL IMPOSTS

Townships	$3,571,200	25.6	$3,519,500	25.3	$6,831,700	49.1			$13,922,400	100.0	12.8
Places up to 2,500	5,071,400	44.7	1,198,600	10.6	739,700	6.5	$4,326,500	38.2	11,336,200	100.0	10.4
Places 2,500 to 15,000	5,565,900	41.1	1,609,600	11.9	833,000	6.1	5,543,400	40.9	13,551,900	100.0	12.5
Places 15,000 to 75,000	6,468,200	43.3	1,608,300	10.8	268,100	1.8	6,599,100	44.1	14,943,700	100.0	13.8
Places 75,000 to 400,000	1,228,000	41.7	326,700	11.1			1,388,400	47.2	2,943,100	100.0	2.7
Chicago	11,282,100	21.8	3,857,600	7.4			36,768,800	70.8	51,908,500	100.0	47.8
Total	33,186,800	30.5	12,120,300	11.2	8,672,500	8.0	54,626,200	50.3	108,605,800	100.0	100.0
Federal aid	1,089,900								4,089,900		
Loans and reserves	3,144,400	5.9	[2] 4,719,800	8.8	265,000	.5	[3] 45,474,000	84.8	53,603,200	100.0	
Grand total	40,421,100	24.3	16,840,100	10.1	8,937,500	5.4	100,100,200	60.2	166,298,900	100.0	

[1] Of this sum, $2,047,000 represents a balance from the highway bond issues. This amount will be repaid entirely through motor-vehicle imposts.
[2] This amount represents unexpended balances from reserve funds and bond issues and also new borrowings. If the present policies are continued in the future, at least half of this amount will be retired from the counties' share of State motor-vehicle revenues and the balance from local property taxes.
[3] These funds come primarily from new borrowings largely in the form of special assessments which will in all probability be retired through property imposts.

1930. However, information is given in the footnotes of table 27 regarding the provisions for retirement of the loans.

The data given in this table should not be confused with the information regarding street and highway tax levies given in the earlier part of this report, and in the report published by the State of Illinois. Tables 7 and 8, for example, give the total amounts levied for street and highway purposes in 1930. The funds so derived were expended in paying 1930 highway costs, in the retirement of indebtedness, and in the establishment of reserves for future work. Table 27 is concerned only with the amount spent on the current program in 1930. Thus it is shown in table 8 that the townships contributed 11.3 percent of the total street and highway levies in 1930; while table 27 shows that the townships contributed 12.8 percent of the current taxes expended in 1930 on streets and highways.

Property taxes expended for street and highway purposes approximated $63,000,000; 75 percent of this amount was spent on urban streets. Motor-vehicle taxes were expended to the amount of more than $45,000,000, of which approximately 73 percent was spent on State highways. Expenditures from loans and reserves exceeded $53,000,000, with approximately 85 percent of this sum devoted to city and village streets.

It should be noted that the imposts on general property set forth in table 27 include special assessments levied for highway and street construction, almost all of such assessments being in incorporated areas.

STUDY OF SURVEY DATA SHOWS RELATIONSHIPS BETWEEN TAXES, TRAVEL, AND EXPENDITURES

The information given in table 27, when studied with reference to other data given in this report, makes it possible to arrive at definite conclusions regarding the relations existing in Illinois in 1930, between the following factors:

1. Taxes paid for highways by taxpayers in the several classes of local government unit;
2. The funds expended on the several highways systems and the local streets;
3. The relative contributions by motor-vehicle owners in the several classes of local unit to the total travel of Illinois vehicles in the State.

CONCLUSIONS

1. No funds raised from taxes on property are expended on State highways.
2. Rural property pays no tax for urban streets.
3. Of the total tax on rural property expended for highway purposes:
   (*a*) 30.9 percent was expended on county trunk highways;
   (*b*) 69.1 percent was expended on local township roads.

TABLE 28.—*Comparison of taxation and expenditures in Illinois in 1930*

$1,000 in taxes			$1,000 in expenditures				
Residents of—	Pay—	In—	Division by purpose	Subdivision by highway system	Source of expenditure	Amount	Percent
Townships	$96.86	General property taxes, $733.71.	Education, $304.03.	State highways, $73.33	Taxes paid in Townships	$6.48	8.83
Place up to 2,500	36.51				Incorporated place	53.73	73.27
Places 2,500 to 15,000	72.03				Federal aid	7.42	10.12
Places 15,000 to 75,000	85.33				Loans and reserves	5.70	7.78
Places 75,000 to 400,000	14.06						
Chicago	428.92						
Townships	8.29	Motor vehicle taxes, $92.97.	Highways, $301.08	County trunk highways, $30.55.	Taxes paid in Townships	6.39	20.90
Places up to 2,500	12.46				Incorporated places	15.60	51.07
Places 2,500 to 15,000	14.67				Loans and reserves	8.56	28.03
Places 15,000 to 75,000	17.00						
Places 75,000 to 400,000	3.28						
Chicago	37.27						
Townships	17.65	Miscellaneous taxes, $173.32.	Public benefit, $341.60.	Township roads, $16.24	Taxes paid in Townships	12.39	76.44
Places up to 2,500	16.30				Incorporated places	3.24	20.00
Places 2,500 to 15,000	25.90				Loans and reserves	.48	2.96
Places 15,000 to 75,000	24.29						
Places 75,000 to 100,000	3.38						
Chicago	85.80						
			Government, $52.69.	Urban streets, $181.59	Taxes paid in Townships		
					Incorporated places	99.10	54.57
					Loans and reserves	82.49	45.43

4. Of the total tax on urban property expended for street and highway purposes:
   (*a*) 7.2 percent was expended on county trunk highways;
   (*b*) 3.5 percent was expended on local township roads;
   (*c*) 89.3 percent was expended on urban streets.
5. Since, of the total assessed property valuation of approximately $7,719,000,000, 26 percent, or approximately $2,007,000,000 was rural, and 74 percent, or $5,712,000,000, was urban:
   (*a*) Expenditures from property taxes for all highways and streets were at the following rates per $100 of assessed valuation:
      Rural—49.2 cents;
      Urban—93.0 cents.
   (*b*) Expenditures from property taxes for county trunk highways were at the following rates:
      Rural—15.2 cents per $100;
      Urban—6.7 cents per $100.
   (*c*) Expenditures from property taxes for local township roads were at the following rates:
      Rural—34 cents per $100;
      Urban—3.2 cents per $100.
   (*d*) Expenditures from property taxes for urban streets were at the following rates:
      Rural—No tax;
      Urban—83.1 cents per $100.
6. Of the total property taxes expended on all roads:
   (*a*) 10.9 percent was expended on county trunk highways;
   (*b*) 13.8 percent was expended on local township roads;
   (*c*) 75.3 percent was expended on urban streets.
7. Of the total motor vehicle imposts expended on all classes of roads and streets:
   (*a*) Rural motor vehicle owners (residents of townships) paid 8.9 percent, and travel by these same rural owners made up 10.5 percent of the total travel on all classes of roads and streets;
   (*b*) Urban motor vehicle owners paid 91.1 percent, and made up 89.5 percent of the total travel.
8. Of the total motor vehicle imposts expended on all classes of roads and streets:
   (*a*) 72.8 percent was expended on State trunk highways, including portions within urban areas;
   (*b*) 11.5 percent was expended on county trunk highways;
   (*c*) 15.7 percent was expended on urban streets.
9. Of all imposts and taxes expended on all roads and streets:
   (*a*) 30.5 percent was expended on State trunk highways;
   (*b*) 11.2 percent was expended on county trunk highways;
   (*c*) 8.0 percent was expended on local township roads;
   (*d*) 50.3 percent was expended on urban streets.
10. Of all imposts and taxes expended on all roads and streets:
   (*a*) Rural property and motor-vehicle owners paid 12.8 percent, and travel by rural vehicles made up 10.5 percent of the total travel on all classes of roads and streets;
   (*b*) City and village property and motor-vehicle owners paid 87.2 percent, and travel by urban vehicles made up 89.5 percent of the total travel.

FINAL COMPARISON OF TAXES AND EXPENDITURES

As a means of summarizing the general taxations and expenditures in Illinois in 1930, table 28 has been

(Continued on p. 56)

STATE GASOLINE TAXES, 1932

[Compiled for calendar year from reports and records of State authorities]

State	Gross tax assessed prior to deduction of refund	Exemption refund (deducted from gross tax)	Net tax earning on motor vehicle fuel [1]	Other receipts, under tax law (licenses, etc.)	Grand total earning (tax and other receipts)	Disposition of grand total earning, according to law: Collection and administration cost [2]	Construction and maintenance on rural roads: State highways [3]	Construction and maintenance on rural roads: Local roads [3]	State and county road bond payments [4]	On city streets	Other than highway purposes	Tax rate, 1932: Cents per gallon, Jan. 1	Tax rate, 1932: Cents per gallon, Dec. 31	Date of rate change	Gasoline, or other fuel for motor vehicles, taxed: Net gallons taxed	Percent change [5]
Alabama	$7,000,502		$7,000,502	$586	$7,001,088	$17,801	$2,465,037	$3,039,028	$1,479,222			5	6	Nov. 5	136,421,624	−16.1
Arizona	3,479,597	$579,376	2,900,221	699	2,900,920	[6]	1,892,298	1,008,622				5	5		58,004,441	−10.4
Arkansas	5,709,727	544,751	5,164,976		5,164,976	166,663	600,488	1,048,828	[7] 3,348,997			6	6		86,082,940	−22.2
California	40,124,804	3,995,950	36,128,854		36,128,854	[8] 18,381	24,051,724	12,026,782			[9] $31,967	3	3		1,204,295,149	−9.4
Colorado	6,134,473	665,253	5,469,220		5,469,220	60,923	3,785,808	1,460,240		[10] $162,249		4	4		136,730,489	−12.6
Connecticut	4,687,912	3,324	4,684,588	47,924	4,732,512	[11]	4,732,512					2	2		234,229,379	−0.9
Delaware	1,145,986	55,836	1,090,150		1,090,150	[12]	685,471		[13] 404,679			3	3		36,338,331	1.7
Florida	14,508,777		14,508,777	22,930	14,531,707	22,930	6,218,047		[14] 6,218,047		[15] 2,072,683	7	7		207,268,239	−11.8
Georgia	11,938,809		11,938,809		11,938,809	4,200	7,956,406	1,989,102			[16] 1,989,101	6	6		198,980,154	−10.3
Idaho	2,539,950	262,223	2,277,727	9,568	2,287,295	10,892	1,938,510		[17] 328,751		[18] 9,142	5	5		45,554,550	−12.3
Illinois	29,988,421	1,234,370	28,754,051		28,754,051	182,098	19,047,969	9,523,984				3	3		958,468,356	−1.1
Indiana	17,938,367	1,198,806	16,739,561	43	16,739,604	77,557	12,496,546	3,124,126		[19] 1,041,375		4	4		418,489,040	−7.2
Iowa	10,693,343	1,723,206	8,970,137		8,970,137	157,520	3,713,837	3,498,780	[14] 1,600,000			3	3		299,004,568	−17.9
Kansas	10,204,096	2,783,601	7,420,495		7,420,495	[19]	5,620,395	1,800,000				3	3		247,349,852	−8.1
Kentucky	8,202,889		8,202,889	3,216	8,206,105	40,858	8,165,247					5	5		164,057,785	−6.9
Louisiana	8,300,840	118	8,300,722		8,300,722	62,000	3,201,453		3,377,125		[20] 1,660,144	5	5		166,014,436	−11.7
Maine	4,397,400	190,698	4,206,702	[21] 47,674	4,254,376	21,898	2,116,239	2,116,239				4	4		105,167,540	−4.0
Maryland	7,902,161	401,929	7,500,232		7,500,232	12,600	5,930,106			1,482,526	[22] 75,000	4	4		187,505,794	0.9
Massachusetts	16,805,808	286,530	16,519,278		16,519,278	[23] 50,000	15,429,374				[24] 1,039,904	3	3		550,642,607	−1.4
Michigan	21,730,941	1,299,613	20,431,328	29,283	20,460,611	[25] 216,268	12,879,773	4,335,287	3,000,000		[26] 29,283	3	3		681,044,263	−6.4
Minnesota	11,352,359	1,351,802	10,000,557		10,000,557	[27]	6,667,038	3,333,519				3	3		333,351,913	−9.7
Mississippi	6,071,654	421,493	5,650,161	[28] 193,589	5,843,750	[29] 37,650	2,822,009	2,429,935	339,053		[30] 215,103	5½	6	June 1	96,732,445	−16.4
Missouri	9,183,199	233,506	8,949,693		8,949,693	57,850	8,891,843					2	2		447,484,670	−2.8
Montana	3,421,504	731,348	2,690,156		2,690,156	31,601	2,513,150		56,134		[31] 89,271	5	5		53,803,120	−10.9
Nebraska	7,893,113	83,648	7,809,465		7,809,465	15,000	5,845,849	1,948,616				4	4		195,236,623	−14.1
Nevada	868,091	140,974	727,117		727,117	[32]	727,117					4	4		18,177,920	−6.5
New Hampshire	2,710,386	71,545	2,638,841		2,638,841	[33]	1,979,131		659,710			4	4		65,971,040	−0.7
New Jersey	20,963,688	4,346,263	16,617,425	[34] 57,246	16,674,671	29,360	7,295,311		1,260,000	5,000,000	[35] 3,090,000	3	3		553,914,175	−3.0
New Mexico	2,362,264	170,011	2,192,253	18,249	2,210,502	32,610	1,007,724		1,170,168			5	5		43,845,055	−17.7
New York	43,690,844	1,217,157	42,473,687	106,906	42,580,593	[36] 50,000	[37] 9,415,420	5,947,745			[38] 27,167,428	2	3	Mar. 1	1,485,127,929	−2.8
North Carolina	14,124,630	220,984	13,903,646	3,731	13,907,377	8,756	2,188,465	4,742,600	[39] 6,738,538		[40] 229,018	6	6		231,727,434	−7.2
North Dakota	2,842,042	1,006,330	1,835,712	1,439	1,837,151	25,000	1,208,101	604,050				3	3		61,190,398	−9.6
Ohio	36,123,658	1,854,479	34,269,179		34,269,179	140,215	19,072,285	8,603,817		6,452,862		4	4		856,729,484	−12.9
Oklahoma	9,661,097		9,661,097	21,345	9,682,442	81,609	7,184,739	2,394,913			[41] 21,181	4	4	[42] Jan. 1	241,527,434	−4.3
Oregon	6,315,052	723,877	5,591,175		5,591,175	17,045	3,618,680		[43] 1,955,450			4	4		[44] 140,066,134	−9.7
Pennsylvania	30,289,915		30,289,915	511,000	30,800,915	[45] 997,800	21,882,999	5,056,398	2,863,718			3	3		1,009,663,827	−6.7
Rhode Island	2,020,740	166,715	1,854,025	3,686	1,857,711	[46]	855,995	716,385	285,331			2	2		92,701,236	−2.0
South Carolina	6,261,560	36,633	6,224,927		6,224,927	[47]	1,590,469	1,037,488	[48] 3,596,970			6	6		103,748,781	−14.1
South Dakota	4,174,644	1,211,296	2,963,348		2,963,348	[49] 41,950	2,921,398					4	4		74,083,694	−12.7
Tennessee	12,185,360		12,185,360		12,185,360	81,292	3,285,390	3,458,305	[50] 3,631,220		[15] 1,729,153	7	7		174,076,575	−15.8
Texas	30,071,589	3,007,831	27,063,758		27,063,758	[12]	18,094,642		[14] 2,203,177		[51] 6,765,939	4	4		676,593,941	−11.3
Utah	2,174,318	2,406	2,171,912	757	2,172,669	5,317	2,167,352		[52]			4	4		54,297,788	−10.0
Vermont	1,874,648		1,874,648		1,874,648	[53]	1,589,648		285,000			4	4		46,866,212	−4.7
Virginia	11,484,414	674,814	10,809,600		10,809,600	[54]	7,566,720	3,242,880				5	5		216,191,996	−5.6
Washington	12,329,201	1,282,691	11,046,510		11,046,510	[33]	8,837,208	2,209,302				5	5		220,930,195	−9.7
West Virginia	5,184,836	243,045	4,941,791	7,609	4,949,400	15,626	834,106		4,099,668			4	4		123,544,775	−8.3
Wisconsin	16,346,591	1,398,171	14,948,420		14,948,420	[55] 41,550	7,846,463	3,022,477	[14] 1,713,404	[10] 597,360	[56] 1,727,166	4	4		373,710,495	−13.4
Wyoming	1,418,145		1,418,145		1,418,145		951,639	354,506	112,000			4	4		35,453,612	−10.6
District of Columbia	2,053,901	18,404	2,035,497	4,181	2,039,678					2,039,678		2	2		101,774,858	17.9
Total			513,047,239	1,091,661	514,138,900	2,832,820	301,788,231	94,073,954	50,726,362	16,776,050	47,941,483	Weighted average rate, 3.60 cents			14,250,173,296	−7.5

[1] Net gasoline tax earned after deduction of refunds allowed by law.
[2] Many States pay collection cost from other State funds, and such are noted. Administration cost includes balances allocated to reserve funds for administrative purposes, and amounts are noted.
[3] Since this table covers the calendar year earnings, but not the actual collections during the year, these columns are not comparable with similar columns in the financial tables, F-1 and F-4 issued by this bureau, which cover different periods fixed by State and local agencies; also certain funds are allocated to bond payments shown in next column.
[4] For State highway bonds, except as noted.
[5] Shows percent increase or decrease (—) compared to net gallons reported in previous year.
[6] Paid from State highway budget, $16,861.
[7] Includes $1,024,069 payments on county road bonds.
[8] Consists of $17,393 for controller's refund division and $988 to employees' retirement fund; excludes $61,857 held in reserve in 1931 for collection expenses in 1932.
[9] For expenses of transportation tax division of board of equalization.
[10] Expended on streets connecting State highways.
[11] From motor vehicle department receipts, $30,000.
[12] Paid from State general fund.
[13] Includes $179,653 payment on county road bonds.
[14] Payments on county road bonds.
[15] For State general fund.
[16] To an equalization fund for public schools.
[17] Payments on State highway treasury notes.
[18] Aviation fund collected from aviation gasoline tax.
[19] Paid from State general fund, $15,000.
[20] Consists of $830,072 for State board of education, and $830,072 for boards of commissioners of ports of New Orleans and Lake Charles Harbor.
[21] Includes receipts from 1-cent tax on gasoline not used in motor vehicles.
[22] To conservation department for oyster propagation.
[23] Estimated.
[24] For Metropolitan District Commission.
[25] Includes $107,430 special legal costs.
[26] Includes $3,615 from dealer's licenses allocated to State general fund, and $25,668 from aviation gasoline tax allocated to aeronautic fund.
[27] Paid by State appropriation, $11,032.
[28] Includes $117,817 receipts from special gasoline tax, levied in three counties for sea wall.
[29] Includes $15,000 allocated to gasoline inspection fund.
[30] For sea-wall protecting road.
[31] Referendum expenses and experimental equipment.
[32] Paid by State tax commission, which collects the gasoline taxes.
[33] Paid by motor vehicle department.
[34] Includes taxes on 411,404 gallons used by motor boats and taxed 2 cents (rebate of 1 cent on 3-cent tax).
[35] Includes $90,000 for Department of Commerce and Navigation, and $3,000,000 for unemployment relief.
[36] Paid to refund reserve; collection cost of $90,780 paid from State general fund.
[37] Portion of appropriation from general funds estimated to have been derived from gasoline taxes.
[38] Includes $11,430,432 for State emergency fund for unemployment relief, $1,486,936 for New York City general fund, and $14,250,059 for State general fund.
[39] Includes $373,424 for county road bonds.
[40] For State revenue department.
[41] Gasoline tax law penalties, allocated to State general fund.
[42] Changed from 5-cent tax of previous year.
[43] Pro rata of gasoline taxes for bond service, balance being from motor vehicle receipts.
[44] Includes approximately 2,294,022 gallons of "distillate" at 3½-cent tax.
[45] Includes portion of State highway patrol expenses, and undistributable items.
[46] Paid from motor-vehicle fees, $15,760.
[47] Paid from State tax commission appropriation.
[48] Includes $3,532,646 for county road bonds.
[49] Includes $8,841 for expenses of State auditor.
[50] Includes $1,729,153 for county road bonds.
[51] For free school fund.
[52] Payments reported from motor-vehicle receipts, instead of pro rating a share from gasoline taxes.
[53] Paid $2,500 from motor vehicle department appropriation.
[54] Paid from State appropriation, $13,400.
[55] Includes $12,300 held in bad checks.
[56] Allotted to county general fund in lieu of personal property tax on motor vehicles.

MOTOR-VEHICLE REGISTRATIONS, 1932 [1]

[Compiled for calendar or registration year from reports of State authorities]

State	1932 registered motor vehicles individually and commercially owned [2]: Grand total registered motor cars and trucks	Passenger automobiles, taxies, and busses [4]	Motor trucks and road tractors	Other registered vehicles: Trailers	Motorcycles	Tax-exempt official motor vehicles and motorcycles [3]: United States cars	State and local cars	Motorcycles (official)	Number of licenses or permits: Dealers	Operators and chauffeurs	1931 grand total registered motor cars and trucks	Year's change in motor-vehicle registration; increase or decrease (−): Motor vehicles: Number	Percent	Passenger cars, percent	Motor trucks, percent	State
Alabama	226,471	194,237	32,234	3,948	604	403	916		2,239	2,558	246,465	−19,994	−8.1	−8.6	−5.1	Alabama.
Arizona	94,947	80,099	14,848	1,765	303	672	1,340	20	136	12,244	105,572	−10,625	−10.1	−13.8	17.5	Arizona.
Arkansas	136,503	112,587	23,916	3,118	188	297	436		275		180,731	−44,228	−24.5	−27.7	−23.5	Arkansas.
California	1,971,616	1,738,385	233,231	61,870	8,338	2,502	17,514	1,034	3,026		2,043,281	−71,665	−3.5	−3.3	−4.9	California.
Colorado	285,860	255,854	30,006	563	805	521	(5)	(5)	2,653	345,311	308,458	−22,598	−7.3	−7.4	−6.5	Colorado.
Connecticut	343,007	289,660	53,347	1,384	2,066	733	2,809	257	2,607	411,485	[6] 354,821	−11,814	−3.3	−4.3	2.1	Connecticut
Delaware	52,851	43,441	[7] 9,410	649	277	149	544	93	559	69,006	55,202	−2,351	−4.3	−3.9	−5.8	Delaware.
Florida	286,091	248,517	37,574	7,522	850	596	2,987	96	2,539	1,132	323,260	−37,169	−11.5	−8.5	−27.4	Florida.
Georgia	287,716	245,666	42,050	4,021	977	698			1,971	3,783	320,840	−33,124	−10.3	−10.5	−9.1	Georgia.
Idaho	95,325	81,993	13,332	9,924	291	263	1,186	15	282	377	111,663	−16,338	−14.6	−14.8	−13.6	Idaho.
Illinois	1,493,498	1,311,783	181,715	8,950	5,274	1,810			3,266	70,788	1,612,770	−119,272	−7.4	−7.0	−9.8	Illinois
Indiana	796,815	675,108	121,707	23,570	2,529	589			2,423	112,496	862,672	−65,857	−7.6	−7.9	−6.3	Indiana.
Iowa	680,330	606,523	73,807	2,833	1,605	424	4,230	64	1,891	154,110	748,438	−68,108	−9.1	−9.5	−5.9	Iowa.
Kansas	504,367	432,610	[7] 71,757	2,014	873	338	3,172	41	1,511	50,486	559,176	−54,809	−9.8	−9.6	−10.8	Kansas.
Kentucky	293,265	260,959	32,306	(8)	774	466	2,600	120	728	9,771	327,326	−34,061	−10.1	−10.7	−7.6	Kentucky.
Louisiana	242,748	198,787	43,961	6,624	773	450			325	19,856	263,050	−20,302	−7.7	−7.7	−8.0	Louisiana.
Maine	171,757	136,774	34,983	4,195	978	229	1,791	67	1,159	287,883	188,238	−16,481	−8.8	−8.5	−9.8	Maine
Maryland	321,242	286,583	34,659	1,327	1,553	1,018		1,553	5,845	74,042	325,372	−4,130	−1.3	−1.0	−3.9	Maryland.
Massachusetts	801,909	698,358	103,551	525	1,661	1,301	4,000	500	3,454	1,014,635	840,190	−38,281	−4.6	−5.2	−0.3	Massachusetts.
Michigan	1,134,808	1,000,169	134,639	77,437	2,996	866			1,553		1,230,980	−96,172	−7.8	−7.2	−11.8	Michigan.
Minnesota	683,397	581,905	101,492	21,839	1,697	644	1,451		1,141	45,104	720,401	−37,004	−5.1	−4.9	−6.4	Minnesota.
Mississippi	149,095	120,180	[7] 28,915	2,594	178	262			2,237		183,650	−34,555	−18.8	−21.4	−5.9	Mississippi.
Missouri	717,460	618,195	[7] 99,265	10,008	1,545	842	1,979	21	1,733	42,096	752,805	−35,345	−4.7	−5.9	3.4	Missouri.
Montana	109,129	88,647	[7] 20,482	61	[7] 399	631	1,410	5	463	335	127,166	−18,037	−14.2	−14.0	−14.8	Montana.
Nebraska	375,716	322,347	53,369	13,531	834	278	1,651		2,266		416,131	−40,415	−9.7	−9.5	−10.8	Nebraska.
Nevada	31,830	25,035	6,795	646	89	116	555		83		32,168	−338	−1.1	−0.7	−2.2	Nevada.
New Hampshire	106,431	88,141	18,290	1,448	1,000	231	397		503	129,429	111,510	−5,079	−4.6	−5.1	−3.0	New Hampshire.
New Jersey	857,850	726,201	131,649	2,863	5,256	872			2,716	1,049,387	869,867	−12,017	−1.4	−1.4	−1.3	New Jersey.
New Mexico	76,767	61,720	15,047	591	233	410	1,050		94		81,325	−4,558	−5.6	−5.7	−5.3	New Mexico.
New York	2,241,930	1,931,384	[7] 310,546	13,796	11,593	3,048	22,326	1,162	4,951	3,145,149	2,297,249	−55,319	−2.4	−1.8	−6.1	New York.
North Carolina	375,695	328,500	47,195	7,774	1,140	563	8,929	75	3,842		428,737	−53,042	−12.4	−12.2	−13.5	North Carolina.
North Dakota	153,570	130,660	22,910	39	230	181	437		394		171,293	−17,723	−10.3	−9.7	−13.8	North Dakota.
Ohio	1,589,524	1,420,550	168,974	45,588	6,763	1,664	12,622	389	3,220	22,129	1,710,625	−121,101	−7.1	−6.5	−12.0	Ohio.
Oklahoma	428,302	379,599	48,703	(8)	[9] 1,020	742					482,725	−54,423	−11.3	−11.3	−10.8	Oklahoma.
Oregon [10]	259,271	[11] 236,405	[12] 22,866	1,706	1,535	538	3,138		415	26,736	278,225	−18,954	−6.8	−7.4	−0.4	Oregon. [10]
Pennsylvania	1,664,021	1,448,978	215,043	7,835	9,744	1,816	1,307	1,375	9,559	2,129,131	1,741,942	−77,921	−1.5	−4.8	−2.2	Pennsylvania.
Rhode Island	133,408	114,950	18,458	68	824	189	910	115	280	163,778	137,878	−4,470	−3.2	−2.8	−5.7	Rhode Island.
South Carolina	177,020	157,453	19,567	2,041	485	290	1,848	163	476	24,228	203,719	−26,699	−13.1	−12.6	−16.5	South Carolina.
South Dakota	161,933	142,552	19,381	8,852	213	342	970	7	500		193,025	−31,092	−16.1	−15.9	−17.6	South Dakota.
Tennessee	298,713	267,279	31,434	3,294	1,087	415	5,760	75	490		350,520	−51,807	−14.8	−15.6	−7.5	Tennessee.
Texas	1,191,324	1,001,675	189,649	35,113	3,194	1,729	11,062	314	2,503	74,094	1,297,301	−105,977	−8.2	−7.8	−10.1	Texas.
Utah	99,851	83,089	16,762	879	434	428					108,958	−9,107	−8.3	−9.1	−4.6	Utah.
Vermont	77,595	69,230	8,365	519	479	138			310	94,329	83,877	−6,282	−7.5	−8.2	−1.0	Vermont.
Virginia	370,587	308,806	61,781	1,673	2,031	1,366	4,735	153	3,267	11,588	379,227	−8,640	−2.3	−4.2	8.4	Virginia.
Washington	446,001	381,490	64,511	4,543	1,776	1,220	6,017	177	1,439		420,878	[13] 25,123	6.0	5.7	7.4	Washington.
West Virginia	227,888	193,232	34,656	1,785	999	162	2,632	168	6,402	59,462	253,308	−25,420	−10.0	−9.7	−11.9	West Virginia
Wisconsin	694,652	587,906	106,746	1,066	2,204	514	5,812	320	1,708		754,249	−59,597	−7.9	−8.2	−6.2	Wisconsin.
Wyoming	56,226	46,330	9,896	(8)	116	332	431		301		62,101	−5,875	−9.5	−9.8	−7.6	Wyoming.
District of Columbia	160,567	142,890	17,677	607	689	1,854	2,712	689	2,899	73,711	173,519	−12,952	−7.5	−8.0	−3.8	District of Columbia.
Total	24,136,879	20,903,422	3,233,457	412,998	91,296	[14] 36,475	143,666	9,068	92,634	9,730,649	[15] 25,832,884	−1,696,005	−6.6	−6.5	−6.7	Total.

[1] This table lists only the number of registrations, licenses, and permits; for financial statement see p. 55.
[2] The first 3 columns show regularly registered motor cars and trucks, eliminating reregistrations, nonresident registrations, tax-exempt cars, etc.
[3] These official cars are exempted from paying regular registration fees and are excluded from "registered motor vehicles".
[4] Busses are included with passenger cars, except as noted in next column.
[5] No exemption of tax on official cars, which are included under registered cars.
[6] Revised figure: Passenger cars revised to 302,594, and motor trucks revised to 52,227.
[7] Busses included with trucks.
[8] Not registered.
[9] Estimate based on 1931 data.
[10] Data covers registration year ending June 30, 1932.
[11] Includes 11,636 light delivery cars used for both passenger and light freight service, and classified by State as passenger cars.
[12] Excludes 11,636 light delivery cars used for both passenger and light freight service.
[13] License fees reduced on motor cars and trucks, January 1, 1932.
[14] Includes 333 cars "at-large", not assignable to any State.
[15] Revised total, see Connecticut as noted above.

MOTOR-VEHICLE RECEIPTS, 1932

[Compiled for calendar or registration year from reports of State authorities]

State	Total gross receipts	Registration receipts [2]					Miscellaneous receipts [2]			Disposition of gross receipts					State
		Motor-car receipts			Other vehicles						For highway purposes				
		Total motor cars	Passenger cars and busses	Trucks and tractors	Trailers	Motor-cycles	Dealers' licenses	Chauffeur and operator permits	Other miscellaneous	Collection and administration [3]	State highways	Local roads	State and county road bonds [4]	For other purposes	
Alabama [2]	$3,038,206									$131,649	$1,010,284	$531,686	$1,364,587		Alabama.[2]
Arizona	709,069	$562,364	$297,457	$264,907	$24,228	$1,046	$1,088	$13,056	$107,287	174,797	534,272				Arizona.
Arkansas	2,796,023	2,770,310					3,229	17,333	5,151	28,063	424,076		[5] 2,343,284		Arkansas
California	9,391,366	8,191,061	5,952,984	2,238,077	345,652	29,692	36,955	172,765	615,241	1,646,481	2,963,648	2,963,647		[6] $1,817,590	California.
Colorado	1,946,983	1,693,157	1,309,997	383,160	7,546	1,477	20,426	182,370	42,007	97,349	924,817	924,817			Colorado.
Connecticut	7,953,950	6,215,370	4,717,373	1,497,997	17,307	8,538	69,009	1,240,403	403,323	[7] 1,032,587	6,921,363				Connecticut
Delaware	1,018,396	866,211	622,079	244,132	8,446	1,073	6,421	103,509	32,736		562,056		[8] 456,340		Delaware.
Florida	5,267,792	5,160,299	4,083,053	1,077,246	43,500	6,211	12,432	2,355	42,995	327,336	2,591	864		[9] 4,937,001	Florida
Georgia	3,825,528	3,746,228	3,015,725	730,503	36,456	4,480	21,611	7,544	9,209	[10]	3,634,251			[11] 191,277	Georgia.
Idaho	1,617,261	1,578,776	1,216,734	362,042	12,370	1,530	16,740	754	7,091	85,273	190,076	1,341,912			Idaho.
Illinois	16,966,682	16,217,286	12,216,109	4,001,177	125,329	16,440	62,420	230,073	315,134		4,954,069	2,500,000	8,362,613	[6] 1,150,000	Illinois
Indiana	6,090,486	5,687,000	1,233,005	1,453,995	128,616	4,640	47,610	34,965	187,655	318,121	5,621,403			[6] 150,962	Indiana
Iowa	11,670,688	10,922,090	9,153,065	1,769,025	35,287	5,710	56,846	78,245	572,510	462,339	10,834,950			[12] 373,399	Iowa
Kansas	5,438,981							22,955	64,118	214,264	3,424,717	1,800,000			Kansas [2]
Kentucky	4,650,638	4,470,278	3,177,247	1,293,031		3,710	21,239	82,534	72,877	397,976	3,672,698	579,964			Kentucky.
Louisiana	4,120,002	3,893,506	3,001,678	891,828	136,998	4,111	16,505	59,396	9,486	77,000	3,712,752		330,250		Louisiana.
Maine [2]	2,956,638									456,552	652,852		1,704,910	[6] 142,324	Maine [2]
Maryland	3,450,386	2,682,681	2,353,713	328,968	35,740	7,587	31,735	195,173	197,470	345,039	2,484,278			[13] 621,069	Maryland.
Massachusetts	6,567,646	3,813,358	2,575,088	1,238,270	33,465	2,219	10,350	2,029,270	678,984	1,518,341	4,206,781		495,496	[14] 347,028	Massachusetts
Michigan	19,845,711	18,137,510	14,091,487	4,046,023	475,472	11,332	46,260	518,983	646,154	851,324	10,512,414	7,000,000	1,082,059	[6] 389,914	Michigan
Minnesota	10,121,697	9,940,872	8,031,572	1,909,300	68,492	5,861	32,385	49,645	24,442	315,210	6,329,517		[15] 3,420,173	[11] 56,797	Minnesota
Mississippi	2,137,953	2,016,736	1,987,821	28,915	2,594	178	11,185		107,260	[16] 181,612	156,789	1,799,552			Mississippi
Missouri	9,824,564									600,439	4,305,098		4,919,027		Missouri [2]
Montana	1,294,174	1,247,022	996,539	250,483	273	784	16,113	667	29,315	[14] 14,127		1,250,047			Montana
Nebraska	3,349,152	3,250,069	2,752,319	497,750	13,186	2,569	30,055		53,273	102,856	973,889	2,272,407			Nebraska
Nevada	395,140	385,808	282,646	103,162	3,607	392	2,696		2,937	30,541	250,060		114,839		Nevada
New Hampshire	2,101,096	1,723,891			[17]	4,786	24,789	282,746	67,884	257,103	1,846,993				New Hampshire
New Jersey	15,413,228	11,567,111	7,573,516	3,993,595	[17]	10,512	67,650	3,153,394	614,561	[18] 347,841	2,376,483	7,310,000		[19] 5,378,904	New Jersey
New Mexico	769,869	734,487	496,994	237,493	6,502	604	5,995		22,281	66,461	316,534	281,363		[6] 105,511	New Mexico
New York	41,272,035	36,732,560	26,576,735	10,155,825	207,856	46,703	206,754	2,966,618	1,111,545	[2] 910,760	13,002,249	9,953,483		[11] 17,405,543	New York
North Carolina	5,444,356	5,361,531	4,316,180	1,045,351		4,409			78,416	370,745	922,854	1,674,908	[21] 2,199,838	[22] 276,011	North Carolina
North Dakota	1,799,973	1,786,486	1,432,986	353,500	514	1,428	10,862		683	73,615	1,019,629	707,329			North Dakota.
Ohio	18,424,972	17,493,326	11,610,960	5,882,366	416,479	23,393	64,400	66,387	360,987	545,317	4,799,769	13,079,886			Ohio.
Oklahoma [2]	4,789,291									104,464	1,954,931	2,729,896			Oklahoma [2]
Oregon	6,548,171	5,942,758	4,890,466	1,082,292	31,532	5,607	20,325	25,690	522,259	374,849	2,248,019	1,473,581	2,172,722	[6] 279,000	Oregon
Pennsylvania	29,815,715	22,313,417	15,401,709	6,911,708	127,178	26,634	235,495	4,400,208	2,712,783	1,638,870	24,305,458		3,180,432	[6] 690,955	Pennsylvania.
Rhode Island	2,183,509	1,767,181	1,328,180	439,001	1,721	3,037	13,140	327,556	70,874	278,193	1,849,125	56,191			Rhode Island
South Carolina	2,470,470	2,233,083	1,823,678	409,405	47,706	1,982	11,800	103,622	72,276	65,900	637,882		[23] 1,592,410	[6] 174,278	South Carolina
South Dakota	2,444,290	2,386,475	1,939,046	447,429	13,415	741	12,340		31,319	69,606	484,000	1,890,684			South Dakota
Tennessee [2]	3,872,094								25,650	125,465	3,554,708			[6] 191,921	Tennessee
Texas	13,154,999	12,333,770	8,556,326	3,777,444	289,483	12,970	32,218	222,321	264,237	741,261	4,010,941	8,402,797			Texas
Utah	801,846									64,346			737,500		Utah
Vermont	2,218,413	1,919,208	1,498,537	420,671	2,792	3,016	29,120	233,959	30,318		2,218,413				Vermont
Virginia	6,240,867	5,869,032	4,862,649	1,006,383	15,396	7,528	55,647	57,864	235,400	385,883	5,854,984				Virginia
Washington	2,179,547	2,001,055	1,169,680	831,375	52,602	4,738	31,270	79,800	10,082	605,628	1,120,589	373,530		[6] 79,800	Washington
West Virginia	4,064,526	3,778,657	2,917,363	861,294	6,566	4,546	30,452	163,346	140,959	184,223	322,803		3,557,500		West Virginia
Wisconsin	10,281,303	10,041,978	7,512,586	2,529,392	70,060	11,620	17,766	15,432	124,447	780,656	3,294,455	[24] 5,065,792	[25] 1,140,000		Wisconsin
Wyoming [2]	676,442										510,442		166,000		Wyoming [2]
District of Columbia	868,086	167,321	142,920	24,401	607	689	2,899	221,133	475,437	120,560				[26] 747,526	District of Columbia
Detailed totals	292,875,448	259,601,319	[27]	[27]	2,844,973	294,523	1,446,232	17,279,116	11,409,285						Detailed totals
Grand totals	324,273,510									17,550,422	155,911,962	75,964,336	39,339,980	35,506,810	Grand totals

[1] Financial data only on this table, for number of registrations, etc., see p. 54.
[2] Complete details not reported.
[3] States for which no figures are shown make State appropriations for administration.
[4] Payments on State highway bonds except as noted.
[5] Includes payments on county bonds of $699,074.
[6] For State highway patrol.
[7] Includes refunds of $218,352.
[8] Includes $202,587, payments on county road bond interest.
[9] For county school fund.
[10] Administrative expense of $89,198 from State general fund.
[11] For State general fund.
[12] Includes $373,399 for county general fund.
[13] For Baltimore streets.
[14] Includes $312,028 for public safety department and $35,000 for motor-vehicle-liability insurance expenses.
[15] Includes $2,996,223 payments on county road bonds.
[16] Includes $38,000 for 1933 tags, advance payments.
[17] Fees included with motor-car receipts.
[18] Excludes $748,555 for administration from special State appropriation.
[19] Includes $5,116,024 for unemployment relief, $196,000 for free bridge commission, $66,880 for traffic commission.
[20] Excludes $1,519,993 appropriation from State general fund.
[21] Includes $111,211 payments on county road bonds.
[22] Includes State highway patrol, and prison-camp expenses for road work, $124,587.
[23] Includes $1,563,707 payments on county road bonds.
[24] Includes $1,149,084 allotment to counties in lieu of personal property taxes on motor vehicles.
[25] Payment on county road bonds.
[26] Includes $105,996 for street signals, and the remainder for streets as appropriated by Congress.
[27] Detailed receipts available for only 38 States and District of Columbia which total as follows: Passenger cars and busses receipts, $190,088,202 and truck and tractor receipts, $65,148,916.

(Continued from p. 51)

prepared. It shows a comparison between the sources of $1,000 in taxes and the manner in which $1,000 was expended, and is based on tables given previously in the report. The comparison, however, is not a direct one, for the $1,000 unit cannot be identical for both taxes and expenditures. If it were desired to make the two sides of the tabulation balance exactly, as in the case of a true balance sheet, it would be necessary to include proceeds from bonds and loans on the left of the double line, and principal payments on the right, as well as to make due allowance for balances carried over and carried forward. The complete figures necessary for such a presentation are not available, so that expenditures as given cannot be said to balance receipts from taxes. It is believed, however, that table 28 gives a helpful picture of the relation between tax money received and actual expenditures in the year 1930.

PRICE INDICES SHOW TREND OF HIGHWAY CONSTRUCTION COSTS

On the cover of Public Roads this month there appears a chart showing the variation of price indices for highway-construction work from 1922 to the close of 1932. For the years from 1922 to 1930, inclusive, the average price index for a given year is plotted directly under the symbol for that year. For 1931 and 1932 the average index for the fourth quarter is given under the symbol for the year.

An extended discussion of the derivation of these indices will appear in a subsequent issue of Public Roads. It may be desirable, however, to remark here that the general index—designated "composite mile"—is an index in which the weightings assigned to the various items for which prices have been used in determining this index, are the amounts of these items which would have been used in building one mile of highway if that mile had contained an average amount of grading and an average footage of structures and had the surfacing been composed of gravel, macadam, concrete, etc., in amounts proportioned according to the average usage of these various surfaces. The base period used in determining both weightings and average base prices is the period 1925 to 1929, inclusive. The general index (composite mile) shows the trend in prices in the highway construction field taken as a whole.

In addition to the general index, three subsidiary indices are shown—excavation (grading), surfacing, and structures. These indices indicate the price changes which have taken place in the more restricted field which each covers.

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ROAD PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D.C. As his office is not connected with the department and as the department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 10 cents.

DEPARTMENT BULLETINS

No. 136D . . Highway Bonds. 20 cents.

No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.

No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.

No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 660D . . Highway Cost Keeping. 10 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys. 20 cents.

No. 265T . . Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS CIRCULARS

No. 62MC . . Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects. 5 cents.

No. 93MC . . Direct Production Costs of Broken Stone. 25 cents.

No. 109MC . . Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters. 10 cents.

MISCELLANEOUS PUBLICATION

No. 76MP . . The results of Physical Tests of Road-Building Rock. 25 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

MARCH 31, 1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$					$	$				$	
Alabama	2.322.7	5.661.042.13	2.830.520.94	35	166.9	105.1	272.0	56.566.34	28.283.17	2.5		2.5	3.195.356.06	Alabama
Arizona	1.219.8	2.694.546.83	1.058.660.39	67	86.0	177.7	263.7	64.714.98	12.942.99	6.6		6.6	112.435.64	Arizona
Arkansas	1.937.4	4.313.869.76	2.039.695.13	45	115.2	62.2	177.4	1.550.273.37	775.081.32	139.9	6.3	146.2	830.315.04	Arkansas
California	2.485.0	8.762.746.88	2.268.772.80	73	152.7	37.0	189.7	1.324.564.46	126.877.45	45.2	4.3	49.5	114.372.99	California
Colorado	1.789.6	3.931.680.35	1.766.500.13	59	163.5	33.2	196.7	312.286.04	140.528.69	26.2		26.2	282.511.30	Colorado
Connecticut	294.8	4.684.457.13	1.825.799.33	63	51.0		51.0						97.238.46	Connecticut
Delaware	369.7	840.301.25	161.712.72	46	25.7	21.9	47.6	53.852.00	14.512.72	3.2		3.2		Delaware
Florida	661.0	6.146.330.95	2.937.024.22	67	169.5		169.5	458.404.35	229.202.17	.5		.5	1.019.854.30	Florida
Georgia	3.162.7	5.805.261.35	2.362.480.22	68	152.1	208.5	360.6	52.296.65	23.533.50	3.7		3.7	32.400.89	Georgia
Idaho	1.534.2	2.640.559.25	988.092.70	72	117.9	131.1	249.0	233.569.15	83.413.50	18.0		18.0	94.738.29	Idaho
Illinois	3.077.7	20.994.544.39	8.138.288.99	80	650.0	36.8	686.8	1.205.435.42	500.544.76	46.0		46.0	40.213.79	Illinois
Indiana	2.066.2	7.951.703.65	3.371.440.29	87	299.1		299.1	1.863.503.92	187.764.78	88.6	5.3	93.9	75.706.46	Indiana
Iowa	3.514.7	5.713.863.68	1.005.785.38	86	286.1	40.0	326.1	29.981.52	380.00		1.2	1.2	40.861.88	Iowa
Kansas	3.839.6	3.996.969.82	1.391.650.02	68	248.5	130.0	378.5	1.247.512.21	254.415.85	82.1	26.0	108.1	218.758.02	Kansas
Kentucky	1.889.0	4.616.147.88	1.647.083.91	53	189.4	138.7	328.1	5.128.59	2.154.00				151.809.31	Kentucky
Louisiana	1.605.8	7.337.649.87	3.211.146.73	59	55.0	26.2	81.2						1.347.11	Louisiana
Maine	818.9	2.387.703.90	524.500.29	76	70.5	.3	70.8	38.958.62	3.989.09		1.9	1.9	129.774.59	Maine
Maryland	828.9	1.526.803.52	455.161.63	85	67.1	2.5	69.6	354.798.40	28.745.00	10.6		10.6	23.469.09	Maryland
Massachusetts	874.2	4.198.799.43	1.013.667.21	57	59.5	4.9	64.4						34.120.04	Massachusetts
Michigan	2.312.0	7.083.949.22	2.803.064.95	61	291.8	114.2	406.0	1.106.862.91	362.100.00	64.9	1.4	66.3	43.626.17	Michigan
Minnesota	4.312.5	4.645.947.47	133.628.91	85	149.6	139.5	289.1	114.649.27	100.00	1.3	.4	1.7	69.532.87	Minnesota
Mississippi	1.818.7	6.914.594.06	3.414.116.39	57	241.1	84.2	325.3	551.221.45	275.610.69	16.9	1.0	17.9	3.052.467.45	Mississippi
Missouri	3.197.3	3.009.747.64	521.702.69	58	131.1	11.0	142.1	2.758.925.23	530.530.92	114.3	17.0	131.3		Missouri
Montana	2.817.1	6.566.112.12	3.681.348.93	58	488.6	268.6	757.2	474.552.61	266.748.90	51.8	23.9	75.7	353.996.52	Montana
Nebraska	4.245.8	6.450.011.64	2.992.718.89	46	171.6	159.8	331.4	101.108.51	42.294.53	8.4	1.6	10.0	5.484.41	Nebraska
Nevada	1.331.9	1.892.977.63	745.827.19	83	29.6	127.3	156.9	315.660.18	9.458.43	29.6	5.9	35.5	103.855.70	Nevada
New Hampshire	434.1	883.346.30	371.757.23	67	20.8	4.1	24.9	110.829.62	46.427.90	2.0	.7	2.7	67.067.62	New Hampshire
New Jersey	632.5	6.084.562.24	2.073.149.85	68	61.0		61.0						96.362.96	New Jersey
New Mexico	2.230.3	3.262.976.53	1.349.645.18	61	190.9	96.6	287.5	321.875.91	151.605.15	17.3	5.2	22.5		New Mexico
New York	3.501.2	18.000.156.60	5.582.840.00	48	476.8	28.5	505.3	1.351.477.65	489.690.00	27.3	3.6	30.9	114.900.01	New York
North Carolina	2.259.2	4.592.771.29	2.295.740.69	50	505.5	23.6	529.1	1.217.931.47	615.722.23	131.0	.9	131.9	1.108.713.32	North Carolina
North Dakota	5.299.0	4.053.860.48	1.658.786.78	49	339.7	395.2	734.9	1.003.259.33	315.658.72	68.7	278.3	347.0	402.041.34	North Dakota
Ohio	3.007.0	9.308.681.31	2.581.506.90	69	210.8	66.8	277.6	251.194.00	93.680.49	4.4	3.3	7.7	172.736.01	Ohio
Oklahoma	2.438.6	4.818.559.21	1.807.645.20	70	209.9	82.8	292.7	1.322.790.34	264.557.92	88.1	8.3	96.4	368.531.42	Oklahoma
Oregon	1.603.1	4.141.177.35	1.715.868.76	59	127.0	83.0	210.0	222.027.85	84.632.43	4.5	.1	4.6	85.450.80	Oregon
Pennsylvania	3.190.5	11.859.519.78	3.409.360.23	53	419.4	14.0	433.4	428.754.95	161.792.80	15.2		15.2	40.772.28	Pennsylvania
Rhode Island	271.7	1.071.153.47	346.631.18	30	22.5	4.5	27.0						46.110.45	Rhode Island
South Carolina	1.957.4	3.522.878.38	1.330.142.73	65	162.8	158.2	321.0	2.411.64	825.00	.1		.1	8.570.16	South Carolina
South Dakota	4.286.0	3.571.429.82	1.504.575.32	72	251.4	194.7	446.1	274.267.06	62.171.28	33.7	20.8	54.5	172.521.89	South Dakota
Tennessee	1.682.2	5.319.838.91	2.659.198.48	37	169.8	64.1	233.9	239.240.16	119.620.07	7.8	3.1	10.9	576.790.79	Tennessee
Texas	7.316.9	19.234.233.26	7.220.307.34	66	759.8	522.0	1.281.8	3.048.676.52	690.793.71	161.7	99.7	261.4		Texas
Utah	1.242.3	1.921.545.76	768.247.53	68	100.5	89.2	189.7	268.098.92	80.923.21	26.5	5.4	31.9	153.359.07	Utah
Vermont	385.8	508.094.30	58.754.29	70	26.6		26.6						18.062.72	Vermont
Virginia	1.925.2	5.008.619.36	2.247.648.33	56	231.0	63.4	294.4	288.394.88	144.197.40	16.9	3.3	20.2	186.278.48	Virginia
Washington	1.329.6	2.587.381.78	815.612.10	51	102.9	2.8	105.7	667.016.85	180.100.00	14.7	7.1	21.8	58.503.33	Washington
West Virginia	925.5	3.035.185.19	1.235.826.15	64	113.6	8.2	121.8	118.441.06	47.441.06	.2		.2	37.499.95	West Virginia
Wisconsin	2.744.8	6.160.663.09	1.197.521.70	71	159.4	101.9	261.3	52.061.50	21.500.00		.9	.9	162.493.57	Wisconsin
Wyoming	2.076.6	3.202.880.00	1.149.934.91	88	268.2	163.3	431.5	274.623.60	27.103.37	27.2	14.5	41.7	40.260.03	Wyoming
Hawaii	78.4	2.760.432.02	1.639.870.83	44	68.0		68.0						508.579.87	Hawaii
TOTALS	105.645.1	265.678.268.23	98.310.962.69	62	9.627.7	4.227.6	13.855.3	25.738.200.49	7.497.655.25	1.407.9	551.4	1.959.3	14.549.852.48	TOTALS
Construction completed		150,542.000.00	60,724.000.00											
Balance uncompleted		115.136,000.00	37,587.000.00											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

A22.6

PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 4 JUNE 1933

FEDERAL-AID HIGHWAY NEAR LENAWEE, MICHIGAN

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

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BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 4 June 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

THE MICHIGAN FINANCIAL SURVEY

Digest of a Survey of the Finances of Michigan in 1930 With Special Reference to Highways, Conducted by the Bureau of Public Roads and the University of Wisconsin

THIS STUDY of highway finance in Michigan is one of a series conducted by the United States Bureau of Public Roads in cooperation with the University of Wisconsin and the respective State highway departments, involving expenditures for highway purposes and the incidence of taxes for these and other purposes. The Michigan survey is based on the calendar year 1930.[1] The following summary of the comprehensive report made by the investigators has been prepared by the Division of Highway Transport of the Bureau in order to make the results available in concise form. The arrangement of material follows that of the Wisconsin and Illinois summaries for greater ease of comparison.[2]

The calendar year 1930 was selected for the study because of its being a United States census year, as well as the latest year for which certain data could be obtained. It is also the same year for which the similar studies in Wisconsin and Illinois were made.

To classify the incidence of taxation, the State was divided into groups of governmental units and into classes of residents. To distribute expenditures, the highway classifications in use in the State were adopted.

Division into groups of governmental units and classes of residents was as follows:

The State Government.

The counties:

Group 1.—All counties with a population of over 400 persons per square mile in 1930.

Group 2.—All counties with a population of from 75 to 400 persons per square mile in 1930.

Group 3.—All counties with a population of from 45 to 74 persons per square mile in 1930.

Group 4.—All counties with a population of from 30 to 44 persons per square mile in 1930.

Group 5.—All counties with a population of from 20 to 29 persons per square mile in 1930.

The municipalities or local governmental units:

1. Townships (rural areas outside of incorporated municipalities).
2. Incorporated places having a population of 2,500 or less.
3. Incorporated places having a population of 2,500 to 15,000.
4. Incorporated places having a population of 15,000 to 75,000.
5. Incorporated places having a population of 75,000 to 400,000.
6. Incorporated place having a population of over 400,000 (city of Detroit only).

Purposes of the investigation were as follows: (1) To study and analyze the direct and indirect highway receipts and disbursements of the State and of the counties, townships, municipalities, and other political subdivisions thereof; (2) to develop facts and information showing the effect of highway and related taxes on property of all kinds; and (3) to develop the total amount of revenues raised for all other governmental purposes of the State, counties, and local units as compared with the revenues raised for highways, bridges, and streets.

The study was undertaken in Michigan partly because of its distinct type of highway development and partly because the financial data pertaining to all governmental units within the State have never before been assembled and analyzed. In connection with the Wisconsin and Illinois surveys it gives a picture of three typical Middle Western States with radically different highway programs.

In these comprehensive investigations many facts pertaining particularly to the subjects investigated and also incidental to the central purposes have been revealed. Some of the conditions noted in Michigan are: (1) Michigan, which started its highway-development program under a system which was highly cooperative among all units of government, has reached a stage where each unit of government is entirely in charge of certain types of highways and entirely divorced from the others; (2) because of the provisions of the Covert road law, special assessments are levied for highway purposes not only in urban areas but also against rural property; (3) practically all portions of the State and county highway systems have some type of surfacing; (4) while there is a tax commission which assembles statistics concerning property taxes and also has jurisdiction over certain other taxes, several different State departments are charged with the administration of a number of important State taxes; (5) there is no central assembly of financial data pertaining to the subdivisions of the State, although the forces of the auditor general make periodic audits of the counties; (6) the records and accounts concerning highways in many counties are inadequate and poorly kept; a uniform system under central control should be installed throughout the State; (7) special assessments have reached the point where "it would seem that special assessments may be levied for almost any purpose and are not only especially burdensome, but in most metropolitan areas they have become so frequent that the payment of same is almost a monthly affair. They seriously impair the credit of any municipality."[3]

GOVERNMENTAL ORGANIZATION DESCRIBED

The State government is of the usual type, with a general assembly composed of two houses, a governor, and other State elective officers, nine executive offices, and other appointive bodies. In Michigan the office of State highway commissioner is elective.

[1] The investigation was directed by Henry R. Trumbower, professor of economics at the University of Wisconsin and economist for the Bureau of Public Roads, assisted by the late A. R. Hirst, engineer, and H. R. Briggs, statistician. The Michigan State Highway Department cooperated in making the facts and data available.

[2] For the summary of the Wisconsin survey, see the April 1933 issue of PUBLIC ROADS, and for Illinois, the May 1933 issue.

[3] From the report of the State commission of inquiry and taxation for 1930.

The counties are under the control of the county board of supervisors. This board of supervisors is composed of a representative from each of the townships within the county and also members from the municipalities. There are usually three departments of the county government. These include, first, the general county government, under the immediate jurisdiction of the elective county officers; second, county highway administration, under the control of the county highway department; and third, the drainage districts, under the control of the county drainage commissioner.

The local government consists of a number of concurrent overlapping bodies. The townships are the rural areas. However, villages are considered as a part of the township in which they are located. Incorporated cities are separate and distinct from the townships. Coextensive with the cities and townships are various highway districts, school districts, sanitary districts, park districts, and other special districts. It has not been the practice in this study to make further division of the units of government other than the township and incorporated places. Those other minor governmental areas have been treated as parts of the larger units which contain them.

THE TAXING SYSTEM

The major Michigan tax is the general property tax, which is used as the primary source of revenue for the local units of government and is largely under their control.

Counties derive most of their funds from the county general property tax levied by the board of supervisors. This tax is paid by the local governmental subdivisions in proportion to the value of each unit as determined by the county board of supervisors. A State property tax is also levied and distributed for collection among the counties upon equalized values as determined by the State board of equalization.

Among other duties the State tax commission assembles data as to total property tax levies and taxable values. From this information it determines the average property tax rate in the State, which it applies to the amount which the commission has determined to be the value of the railroads and similar utilities operating in Michigan. This tax is the only tax which the public utilities pay and is specifically dedicated for school purposes. Certain minor taxes are administered by the commission, but its major duties are as outlined above.

The State revenue system is quite diversified. Besides the general property tax, there are inheritance taxes, bond and mortgage taxes, a malt tax, a severance tax, special corporation taxes, taxes upon insurance companies, and other minor imposts. These are administered by several different State departments.

SOME GENERAL CHARACTERISTICS OF THE STATE DISCUSSED

While in 1930 the population of Michigan was 4,842,325 persons, the concentration was primarily in the southern portion of the State. Over 84 percent of the population is in the southern third of the territory, while 9 percent is in central Michigan, and 6½ percent in the Upper Peninsula. Of the total, 1,245,950 persons are residents of unincorporated territory. Of the remaining 3,596,375 village and city residents, 2,606,335 persons live in cities of over 25,000. Of these, 1,568,662 live in the city of Detroit.

The agricultural and manufacturing industries are also concentrated in the southern portion of the State. Over 90 percent of the total taxable value is in southern Michigan. In the northern portion of the State there is some mining. The many glacier-made lakes in this region provide facilities for fishing and water sports which, with the favorable summer temperatures, make the resort and tourist business a source of large revenues.

Many of the general relationships are very similar to those found to exist in Wisconsin and Illinois. In all three States there are substantial contributions from municipal groups to rural areas. This is particularly true as to the northern portions of Wisconsin and Michigan. In all three States the one large metropolitan center exerts a very decided effect upon the observed tendencies.

The assessed valuation of all taxable property in Michigan in 1930 was $8,447,141,100. Of this amount 21 percent was in the townships, 39.1 percent in incorporated places other than Detroit, and 39.9 percent in Detroit. Details are given in table 1.

TABLE 1.—*Assessed valuation in the townships and incorporated places of Michigan in 1930*

Class of place	Amount	Percent
Townships	$1,773,958,100	21.(
Places up to 2,500	328,474,700	3.9
Places 2,500 to 15,000	780,537,600	9.2
Places 15,000 to 75,000	1,334,306,300	15.8
Places 75,000 to 400,000	855,928,700	10.1
Detroit	3,373,935,600	39.9
Total	8,447,141,000	100.(

THE MICHIGAN HIGHWAY SYSTEM

There were, in 1930, 85,080 miles of rural highways in Michigan. The mileage of urban streets is not known. Of the rural highways, 7,691 miles, or 9 percent, were on the State trunk-line system; 17,175 miles, or 20.2 percent, were county roads; and 60,214 miles, or 70.8 percent, were local, or township, roads. In addition to the rural mileage, the State and county systems include portions within urban areas.

The State highway department has direct control of the trunk-line system, which is subdivided into the Federal-aid system of 4,671 miles and the remaining trunk-line roads, which include approximately 3,000 miles. Together these two groups constitute the State system of through highways. It is almost entirely surfaced. The funds for this State highway system are derived from State imposts upon motor vehicles and from Federal aid. A part of the system was financed from a $50,000,000 State bond issue, which will be retired from the State's share of the vehicle imposts.

The system of county roads is under the complete control of the county highway departments. It is primarily intended to serve county travel. An unusually high percentage of the county roads are surfaced. The county system is financed in part through an allotment of State funds equal to one half of the annual vehicle-weight tax, or registration fee, collected, and in part from county property imposts, including both general taxes and special assessments.

Local township authorities have had control over 70.8 percent of all rural highways. Beginning in 1932, these local township roads are all being placed under county control. One fifth of them is to be so transferred annually. By 1937 all rural roads will be under the supervision either of the State or the county highway departments. It is interesting to note that of the

total rural highway mileage, about 10 times the mileage under the control of the State highway department is under the jurisdiction of local authorities.

Few facts are available as to city and village streets in the State of Michigan. Little is known as to their total mileage or their types of surface. It is known that out of the $105,177,100 expended on all highways in Michigan in 1930, $37,502,400, or 35.7 percent of the total of all highway expenditures of the year, were local expenditures on municipal streets. Of this amount, $26,300,200 was expended by the city of Detroit.

THE METHOD OF OBTAINING FINANCIAL DATA

Since in Michigan there is, with the exception of certain facts as to property valuations and general tax levies, no central assembly of county, township, and municipal financial data, it was necessary in the conduct of this survey to obtain data directly from these governmental subdivisions themselves. Michigan statutes do provide that every governmental subdivision must annually publish a complete financial report covering all transactions of the preceding year. These published reports were obtained for the year most nearly corresponding to that of the study. Any differences resulting in the use of such fiscal year reports as compared with the calendar year are probably less than would have been the case if attempts had been made to adjust the fiscal year statements to the calendar year base. With few exceptions, adequate reports were received from the counties and all of the larger cities. Because of the importance of the communities and the complexities of the local records, the statistics for Wayne County and the city of Detroit were obtained directly at the proper local offices in these places. Field visits were also made by members of the staff to other counties and cities in cases where adequate data could not be obtained from the annual reports. The office of the State auditor general also makes periodic audits of all counties. These reports furnished a valuable additional source of information.

For the villages, townships, and small cities, the data were obtained through correspondence. Local officials evinced a willingness to cooperate with the survey, with the result that the necessary statements were provided by more than 60 percent of these minor governmental subdivisions. As there is a close correlation in these units of government between tax levies and expenditures, and as the tax levies were known, it was possible to estimate the facts for the remaining townships and villages with reasonable accuracy.

The State expenditures were taken directly from the books of the State auditor general. The State highway expenditures for construction and maintenance and the allocation of these amounts between townships and incorporated places were prepared for this study by the State highway department.

TAXES LEVIED AND INCIDENCE OF TAXATION

In 1930 the total of all imposts charged against persons and property in Michigan was $384,732,300. The amount of these imposts imposed by authority of each of the governmental agencies is shown in table 2. All of the State and county imposts and taxes are ultimately paid by taxpayers in the townships and incorporated places. The amounts shown in table 2, restated to show the distribution as ultimately payable, are given in table 3. The totals are also subdivided into general property taxes, motor-vehicle imposts, and other taxes.

While general property taxes are the chief source of revenue in Michigan, the State has a variety of other taxes. The amount provided by each major form of tax or impost is shown in table 4.

MICHIGAN TAXES IN 1930

Total taxes and imposts levied in Michigan in 1930 were $384,732,300. Taxation of general property totaled $282,210,900; motor-vehicle fees and motor-fuel taxes, $44,237,100; other sources, including insurance and inheritance taxes, valuation tax on railroads, corporation imposts, certain local imposts, and miscellaneous State revenues, $58,284,300.

Of the total taxes and imposts, taxpayers in the townships contributed $69,729,800, or 18.1 percent; those in incorporated places having a population of 2,500 or less, $18,641,700, or 4.8 percent; in places of 2,500 to 15,000 population, $45,356,600 or 11.8 percent; in places of 15,000 to 75,000 population, $62,360,900, or 16.2 percent; in places of 75,000 to 400,000 population, $39,906,300, or 10.4 percent; in Detroit, $148,737,000, or 38.7 percent.

The average actual tax rates on general property, per $100 valuation, were as follows: Townships, $2.78; places of 2,500 or less population, $3.82; places of 2,500 to 15,000 population, $4.18; places of 15,000 to 75,000 population, $3.52; places of 75,000 to 400,000 population, $3.43; Detroit, $3.30.

Motor-vehicle owners in the townships paid in license fees and gasoline taxes an average of $25.42; in places of 2,500 or less population, the average was $30.20; in places of 2,500 to 15,000 population, $30.62; in places of 15,000 to 75,000, $31.86; in places of 75,000 to 400,000 population, $32.03; in Detroit, $35.49.

TABLE 2.—*Total of all State and local charges imposed by the States and its subdivisions for collection in 1930*

Imposed by	Amount	Percent	Per capita
State	$112,620,900	29.3	$23.26
Counties	54,625,100	14.2	11.28
Townships	29,612,300	7.7	23.79
Places up to 2,500	9,555,700	2.5	32.47
Places 2,500 to 15,000	26,397,000	6.9	47.08
Places 15,000 to 75,000	37,461,300	9.7	56.45
Places 75,000 to 400,000	21,212,400	5.3	50.01
Detroit	90,214,300	23.4	57.51
Total	381,732,300	100.0	79.45

TABLE 3.—*Total of all State and local charges imposed in 1930 as payable by taxpayers in the local units of government*

Payable by taxpayers in—	General property taxes	Motor-vehicle and motor-fuel taxes	Other taxes	All taxes	Percent	Per capita
Townships	$49,383,500	$8,967,300	$11,379,000	$69,729,800	18.1	$55.97
Places up to 2,500	12,547,000	3,212,700	2,882,000	18,641,700	4.8	63.34
Places 2,500 to 15,000	32,633,200	5,506,300	7,217,100	45,356,600	11.8	77.46
Places 15,000 to 75,000	46,989,500	6,194,100	9,177,300	62,360,900	16.2	93.96
Places 75,000 to 400,000	29,373,700	5,026,000	5,506,600	39,906,300	10.4	82.42
Detroit	111,284,000	15,330,700	22,122,300	148,737,000	38.7	94.82
Total	282,210,900	44,237,100	58,284,300	384,732,300	100.0	79.45

TABLE 4.—*Sources of revenue by type of tax or revenue in 1930*

Type of tax	Amount	Percent	Per capita
Ad valorem general taxes	$266,776,400	69.4	$55.09
Special assessments [1]	15,434,500	4.0	3.19
Public utility taxes	14,253,800	3.7	2.94
State motor-vehicle imposts	44,237,100	11.5	9.14
Bond and mortgage taxes	1,909,300	.5	.39
Other local imposts	19,400,500	5.1	4.01
Inheritance taxes	5,932,200	1.6	1.23
Insurance taxes	3,879,300	1.0	.80
Corporation imposts	7,452,800	1.9	1.54
Miscellaneous State revenues	5,456,400	1.3	1.12
Total	384,732,300	100.0	79.45

RECAPITULATION

General property taxes	$266,776,400	69.4	$55.09
Special assessments [1]	15,434,500	4.0	3.19
Motor-vehicle imposts	44,237,100	11.5	9.14
Other local receipts	19,400,500	5.1	4.00
Other State receipts	38,883,800	10.0	8.03

[1] For highways only.

It is of interest to note the relationship between the revenues raised from property taxation and from other imposts. In Michigan, quite consistently, about 73 percent of the total funds come from property imposts. Table 5 gives the rates per $100 of taxable property needed to raise all funds by property tax levies and the actual property tax as it was levied.

TABLE 5.—*Comparison between a rate on general property necessary to produce all revenues from property taxes and the actual tax rate on general property as levied, per $100 valuation*

Class of place	Actual tax rate on general property as levied	Tax rate needed to raise all funds by general property tax levies	Percentage that general property taxes are of all taxes and imposts
Townships	$2.78	$3.93	70.8
Places up to 2,500	3.82	5.68	67.3
Places 2,500 to 15,000	4.18	5.81	71.9
Places 15,000 to 75,000	3.52	4.67	75.4
Places 75,000 to 400,000	3.43	4.66	73.6
Detroit	3.30	4.41	74.8
Average for State	3.34	4.56	73.4

STREET AND HIGHWAY REVENUES DERIVED FROM GENERAL PROPERTY AND MOTOR VEHICLE TAXES

Of the total 1930 tax bill of $384,732,300, the sum of $111,853,900 was levied for street and highway purposes. Table 6 gives the allocation of highway taxes by type of tax.

TABLE 6.—*Taxation imposed for street and highway purposes in 1930*

Type of tax	Amount	Per-cent	Per-cent-age of total	Per capita
On general property:				
State	$40,600	0.1		$0.01
County	19,248,000	28.5		3.97
Local	32,893,700	48.6		6.79
Special assessments	15,434,500	22.8		3.19
Total	67,616,800	100.0	60.5	13.96
On motor vehicles:				
Registration fees	21,247,800	48.0		4.39
Fuel taxes	21,641,200	48.9		4.47
Other fees	1,348,100	3.1		.28
Total	44,237,100	100.0	39.5	9.14
Grand total	111,853,900		100.0	23.10

Levies on general property, including special assessments, total $67,616,800, of which $39,547,400 was imposed specifically for city streets and $8,821,400 for rural roads. A total tax of $19,248,000 was levied on general property by the counties, for use on the county highway systems, including portions within incorporated places.

Imposts on motor vehicles and motor fuel totaled $44,237,100. The money so raised is spent on the State and county systems, including portions of these systems within incorporated places.

Like all other taxes, these street and highway levies are ultimately paid by taxpayers in the townships and urban communities. Table 7 shows the imposts of each type for streets and highways as they are charged to taxpayers in the several classes of place.

MOTOR-VEHICLE REGISTRATIONS, TAXES, AND TRAVEL DISCUSSED

There were 1,378,214 motor vehicles of all types registered in the State of Michigan in 1930, or 1 for every 3.1 persons. There was a passenger car for every 4.1 persons and a truck or bus for every 22.8 persons.

It was important to allocate as accurately as possible the motor vehicles among the local units of government. Data was obtained from the motor vehicle license department and also allocations made by the State highway department. In addition, this material was supplemented by a study of a sample of 20,000 car registrations. The situs of cars of uncertain location was verified by correspondence. From the facts so ascertained a reasonable allocation was made by place of ownership of all motor vehicles among the various classes of local governmental units. These data, divided between passenger automobiles and trucks and busses are given in table 8.

In 1930, Michigan vehicles paid registation fees based upon weight amounting to $21,247,800. In addition to the registration fees there are allied fees such as car registration fees, drivers' licenses, chauffeurs licenses and transfer fees, administered by the motor vehicle licensing department which brought in revenues of $1,147,400. The total cost of the entire department for all of these services was $1,069,200. This amount was 4.77 percent of all the fees received by the department, and averaged 78 cents per motor vehicle registered. As shown in table 8, the average license fees tend to increase fairly steadily as the place of residence becomes more urban. This is true for trucks and busses as well as automobiles. While the average fee per motor vehicle was $15.42, the fee for motor trucks and busses was $24.52, as compared with $13.77 for passenger cars.

GASOLINE-TAX PAYMENTS DETERMINED BY QUESTIONNAIRE METHOD

The 3-cent motor fuel tax in Michigan in 1930 produced a net revenue, after refunds, of $21,641,200. The cost of collection was $41,400, or about $1.91 per $1,000 collected.

To find the gasoline tax paid by vehicles owned in the various groups of places, questionnaires were sent to 50,000 motor-vehicle owners in the State. In order to obtain a fair cross section, this sample was proportioned to the motor-vehicle ownership within the various places. This survey was conducted during the last months of 1931 and the first part of 1932. Since it was impossible to expect intelligent answers from motor-vehicle operators except as to operations in the year immediately preceding, the requests for data on

TABLE 7.—*Incidence of street and highway taxation*

Payable by taxpayers in—	Type of impost											
	Motor-fuel tax		Registration fees		Other motor-vehicle imposts		Special assessments		Property taxes		Total	
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Townships	$4,115,900	19.0	$4,502,800	21.2	$348,600	25.9	$2,150,000	13.9	$13,216,400	25.3	$24,333,700	21.8
Places up to 2,500	1,689,100	7.8	1,443,200	6.8	80,400	6.0	404,500	2.6	2,904,000	5.6	6,521,200	5.8
Places 2,500 to 15,000	2,741,400	12.7	2,602,400	12.2	162,500	12.0	1,650,400	10.7	5,487,000	10.5	12,643,700	11.3
Places 15,000 to 75,000	3,056,800	14.1	2,952,000	13.9	185,300	13.7	2,343,700	15.2	7,035,600	13.5	15,573,400	13.9
Places 75,000 to 400,000	2,470,300	11.4	2,420,900	11.4	134,800	10.0	1,960,400	12.7	3,888,600	7.5	10,875,000	9.7
Detroit	7,567,700	35.0	7,326,500	34.5	436,500	32.4	6,925,500	44.9	19,650,700	37.6	41,906,900	37.5
Total	21,641,200	100.0	21,247,800	100.0	1,348,100	100.0	15,434,500	100.0	52,182,300	100.0	111,853,900	100.0

TABLE 8.—*Motor-vehicle registrations and license fees in 1930*

Place of ownership	Registration	Percentage of total	Persons per vehicle	Registration fees	Percentage of total	Payments per vehicle
Townships:						
Automobiles	273,044	23.4	4.6	$3,419,100	21.3	$12.52
Trucks and busses	65,906	31.1	18.9	1,083,700	20.9	16.44
Total	338,950	24.6	3.7	4,502,800	21.2	13.28
Places up to 2,500:						
Automobiles	84,492	7.2	3.5	1,095,800	6.8	12.97
Trucks and busses	19,225	9.1	15.3	347,400	6.7	18.07
Total	103,717	7.5	2.8	1,443,200	6.8	13.91
Places 2,500 to 15,000:						
Automobiles	147,252	12.6	4.0	2,006,200	12.5	13.62
Trucks and busses	27,276	12.9	21.5	596,200	11.5	21.86
Total	174,528	12.7	3.4	2,602,400	12.2	14.91
Places 15,000 to 75,000:						
Automobiles	165,193	14.2	4.0	2,334,100	14.5	14.13
Trucks and busses	23,440	11.1	28.3	617,900	11.9	26.36
Total	188,633	13.7	3.4	2,952,000	13.9	15.65
Places 75,000 to 400,000:						
Automobiles	132,656	11.4	3.7	1,876,400	11.7	14.14
Trucks and busses	20,055	9.5	24.1	544,500	10.5	27.15
Total	152,711	11.1	3.5	2,420,900	11.4	15.85
Detroit:						
Automobiles	363,978	31.2	4.3	5,327,100	33.2	14.64
Trucks and busses	55,697	26.3	28.2	1,999,400	38.5	35.90
Total	419,675	30.4	3.7	7,326,500	34.5	17.46
State totals:						
Automobiles	1,166,615	84.6	4.2	16,058,700	75.6	13.77
Trucks and busses	211,599	15.4	22.8	5,189,100	24.4	24.52
Total	1,378,214	100.0	37.0	21,247,800	100.0	15.42

travel and gasoline consumed were worded to cover the calendar year 1931. It was presumed that the travel, and therefore the gasoline consumption, were proportionately the same in 1930 as in 1931. Usable replies were received from 9,710 owners. The data so obtained is summarized in table 9.

The indicated total gallonage of gasoline consumed in the year, based on the replies of the operators, is approximately 35 percent greater than that computed on the basis of the tax-paid gallonage. This excess was surprising in view of the fact that exactly similar data obtained from questionnaires in both Illinois and Wisconsin give an indicated gallonage much more nearly approximating that upon which the tax was actually paid. There may be three causes for this difference. The first is that it is a possibility that those driving the shortest distances failed to answer the questionnaire, while on the contrary, those having high mileages would tend to report the fact. The second is that exemption is claimed for gasoline supposedly purchased for nonhighway use, but which actually was used in motor vehicles. The third cause is that there

TABLE 9.—*Average miles traveled and average gasoline consumption in 1931 by place of ownership*

Place of ownership	Number of reports tabulated			Average gasoline consumption			Average miles traveled		
	Automobiles	Trucks and busses	All vehicles	Automobiles	Trucks and busses	All vehicles	Automobiles	Trucks and busses	All vehicles
				Gallons	*Gallons*	*Gallons*			
Townships	1,977	303	2,280	508	712	535	7,966	8,686	8,062
Places up to 2,500	851	91	942	625	1,216	682	9,258	13,438	9,662
Places 2,500 to 15,000	1,386	138	1,524	655	997	686	9,304	10,668	9,427
Places 15,000 to 75,000	1,218	87	1,305	667	1,181	701	9,909	10,550	9,952
Places 75,000 to 400,000	1,088	82	1,170	699	932	716	10,142	10,130	10,141
Detroit	2,404	85	2,489	731	1,351	752	10,389	12,513	10,461
Total	8,924	786	9,710	647	963	673	9,480	10,354	9,551

is undoubtedly evasion of motor-fuel taxes in Michigan as well as in other States of an unknown amount. There is also to be considered the fact that the questionnaire replies are for 1931, in which year the total sale of motor fuel was greater than in the preceding year. It was assumed in this survey that while the motor-vehicle mileage as ascertained by this method was undoubtedly higher than the average travel in the State, the percentage of error would tend to be uniform throughout the sample, since a representative sample was taken. Therefore, proportionate reductions were made in all computations of gasoline taxes paid, in order to reduce the figures to the actual motor-vehicle fuel imposts known to have been paid.

The per vehicle payments of motor-fuel taxes, computed on the basis outlined above, were as follows:

Vehicles owned in—	
Townships	$12.14
Places up to 2,500	16.29
Places 2,500 to 15,000	15.71
Places 15,000 to 75,000	16.21
Places 75,000 to 400,000	16.18
Detroit	18.03
Average for State	15.70

Because urban cars tend to travel greater mileages and also, as indicated by the license fees paid, are, on the average, heavier than rural cars, the gasoline taxes paid in urban communities are higher than those paid in the rural districts.

The data given in table 9 show that the average automobile consumed only 67 percent as much gasoline as was the average for trucks and busses. This fact is reflected in the gasoline-tax payments, the average for automobiles being $14.39, as against $22.96 for trucks and busses. Passenger automobiles paid 77.6 percent of the total gasoline-tax payments; trucks and busses, 22.4 percent.

The total payments of motor-fuel taxes and vehicle licenses, as well as other motor-vehicle imposts, by motor-vehicle owners in the several classes of local unit were given in table 7. The State highway program is financed entirely from these motor-vehicle imposts plus Federal aid. An amount equal to one half of the total vehicle-license fee or weight tax is returned to the counties. Seven eighths of this fund is distributed in proportion to the fees derived from each county and the other one eighth is allocated equally among the 83 counties.

TABLE 9 AFFORDS BASIS FOR ESTIMATING RELATIVE TRAVEL BY RURAL AND URBAN VEHICLES

The data given in table 9 cannot very well be used to estimate the total travel of Michigan vehicles in 1930, since the indicated mileages and gasoline consumption so greatly exceed figures which are based on the actual gasoline-tax payments. The estimate of total travel given in the Michigan transportation survey [4] was based on the taxed gallonage and a figure for mileage per gallon obtained from an analysis of the questionnaire data given in table 9 of this report.

It is reasonable to assume, however, that the errors of the questionnaire are proportional throughout the State; and that the relative travel in 1930 by vehicles owned in the several classes of place was substantially as is indicated in table 9. On this assumption the average mileages given in that table have been applied to the registration figures of table 8, to obtain the percentage of total traffic of Michigan vehicles in 1930 contributed by motor vehicles owned in each class of place. These percentages were as follows:

Percentage of total travel contributed by vehicles owned in—	
Townships	20.6
Places up to 2,500	7.8
Places 2,500 to 15,000	12.4
Places 15,000 to 75,000	14.1
Places 75,000 to 400,000	11.6
Detroit	33.5
Entire State	100.0

It was also found by the same method of computation that passenger automobiles contributed 83 percent of the total travel and trucks and busses 17 percent.

DATA ON MOTOR-VEHICLE TAXES AND TRAVEL SUMMARIZED

In table 10 the payments by motor-vehicle owners in each class of local unit or place are given in both percentage and per-vehicle forms. Percentages of population, motor vehicles registered, and travel are also given for purposes of comparison. The tendency of vehicle imposts to be greatest in the places with the greatest densities in population is decided. Thus the townships, with 24.6 of the registered motor vehicles, paid only 20.3 percent of the motor-vehicle taxes; while Detroit, with 30.4 percent of the registration, paid 34.6 percent of the motor-vehicle taxes.

From table 9 and data previously given, the following conclusions have been derived regarding the taxation and travel of motor vehicles in Michigan in 1930.

1. The townships, with 25.7 percent of the population, contained 24.6 percent of the registered motor vehicles; motor-vehicle owners in townships paid 20.1 percent of the motor-fuel taxes and registration fees, and contributed 20.6 percent of the total travel of Michigan vehicles.

[4] A Survey of Highway Transportation in Michigan, PUBLIC ROADS, vol. 13, no. 12, February 1933, p. 190.

TABLE 10.—*Comparison of the several classes of local gov unit as to motor-vehicle imposts paid, population, nu vehicles registered, and share of total State traffic contrit vehicles owned in each class of place*

Class of place	Motor-fuel taxes paid	Registration fees paid	Motor-fuel taxes and registration fees paid	All motor-vehicle imposts paid [1]	Population	Motor vehicles registered
Townships:						
Percent	19.0	21.2	20.1	20.3	25.7	24.6
Per vehicle	$12.14	$13.28	$25.42			
Places up to 2,500:						
Percent	7.8	6.8	7.3	7.3	6.1	7.5
Per vehicle	$16.29	$13.91	$30.20			
Places 2,500 to 15,000:						
Percent	12.7	12.2	12.5	12.4	12.1	12.7
Per vehicle	$15.71	$14.91	$30.62			
Places 15,000 to 75,000:						
Percent	14.1	13.9	14.0	14.0	13.7	13.7
Per vehicle	$16.21	$15.65	$31.86			
Places 75,000 to 400,000:						
Percent	11.4	11.4	11.4	11.4	10.0	11.1
Per vehicle	$16.18	$15.85	$32.03			
Detroit:						
Percent	35.0	34.5	34.7	34.6	32.4	30.4
Per vehicle	$18.03	$17.46	$35.49			
Entire State:						
Percent	100.0	100.0	100.0	100.0	100.0	100.0
Per vehicle	$15.70	$15.42	$31.12			

[1] For motor-vehicle imposts other than motor-fuel taxes and registration f table 7.

2. Incorporated places other than Detroit, with percent of the population, contained 45 percent of registered motor vehicles; motor-vehicle owners in t places paid 45.2 percent of the motor-fuel taxes registration fees, and contributed 45.9 percent of total travel of Michigan vehicles.

3. Detroit, with 32.4 percent of the population, tained 30.4 percent of the registered motor vehi motor-vehicle owners in Detroit paid 34.7 percen the motor fuel taxes and registration fees, and tributed 33.5 percent of the total travel of Michi vehicles.

4. Of all imposts on motor vehicles, the owner motor vehicles in the townships paid 20.3 perc those in incorporated places other than Detroit 45.1 percent; and those in Detroit paid 34.6 percen

5. The average registration fee paid was $15.42; average motor fuel tax payment, $15.70; the t $31.12. Both registration fees and gasoline-tax ments were found to increase as the place of resid becomes more urban.

6. The average registration fee paid by passe automobiles was $13.77; by trucks and busses, $24 The average motor-fuel tax payment by passe automobiles was $14.39; by trucks and busses, $22 Thus, the average payments in registration fees gasoline taxes were: Passenger automobiles, $28 trucks and busses, $47.48.

7. The owners of motor trucks and busses, w constituted 15.4 percent of the total registrations contributed 17 percent of the total travel of Michi vehicles, paid 24.4 percent of the registration fees 22.4 percent of the motor-fuel taxes.

SPECIAL ASSESSMENTS AND THEIR PLACE IN THE HIGHWA PROGRAM

In Michigan many public improvements are finan through special assessment. In the cities not onl almost the entire cost of street improvements and s lar public works such as sewers and water mains cust arily met through special assessments, but such imp

re made for fire protection, public buildings, and other purposes. A special legislative committee in its report on Michigan taxes states, "One of the most burdensome taxes in Michigan involving our municipalities is that known as 'special assessment taxes.' It would seem that special assessments may be levied for almost any purpose and are not only especially burdensome, but in our metropolitan areas they have become so frequent that the payment is almost a monthly affair. They seriously impair the credit of any municipality. * * * While there is no legislation that can be enacted that will relieve these taxpayers of their present burdens * * * the State can take action, the effect of which will be to afford some measure of protection for the taxpayers of the State from the pyramiding of special assessment taxes in the future." After suggesting remedial legislation, the committee states that among the effects of the proposed legislation one result would be to "definitely limit the maximum assessment against any particular piece of property and preclude the practical confiscation of the property by the levy of special assessment taxes." [5]

These special assessments in Michigan as well as in most other States are not generally regarded as a part of the municipal financial operations and do not appear normally in any statement of receipts or disbursements. However, exactly the same improvements are financed in some localities from special assessments and in others by general-tax levies. Any financial study which fails to take into consideration the enormous public costs, especially in connection with highway improvements, paid for some of these revenues, fails to present a true and complete picture of the situation. As has been indicated by the statements in the previous paragraph these special assessments may become excessively oppressive.

Not only are special assessments a problem in municipalities, but in Michigan they are also prevalent in rural areas. This is due to the Michigan road district law commonly termed the "Covert road act" and the very similar "drainage act." Under these laws special districts are created for the building of highways and for the establishment of drainage areas.

While there are a number of States which permit the formation of rural special assessment road districts, Michigan apparently is about the only State where extensive use of this method of financing rural roads has been made in recent years.

The intent of the Covert road act is to provide an additional method of constructing and improving public highways where a portion of the cost is paid by special assessments upon the lands benefited. When the owners of 66 percent of the lineal frontage of lands touching any highway desire to improve such highways, they may file application with the county road commissioners for the creation of a special assessment district. The commissioners then may fix the boundary of a special assessment district which may include property not touching the highway contemplated and may, without affording them a voice in the matter, cause property owners removed from the highway to have assessments levied against their property for improvements which they may not have desired. After fixing the district, the commissioners allocate the costs. There are no rules or established procedure for this process. A portion of the entire costs is levied as a special assessment against the property benefited. This may vary from as low as 15 percent of the entire cost up to as high as 65 percent. A part of the cost of each project is charged against the township at large, and another part is charged against the county at large. The proportionate amounts of each of these charges may vary as much as is the case with the amounts charged against the property benefited. There is also no uniformity or system in apportioning the benefits against the property included. Various types of frontage and area bases are used.

The number of these Covert road projects runs into the thousands. In 1930 Berrien County alone had 127 active projects. In some counties intelligent and systematic use of the Covert road procedure has resulted in a good system of improved, connected roads serving local traffic. This is one of the reasons for the high percentage of local surfaced roads in the State of Michigan. In other counties, particularly in the Detroit area, an overdevelopment has resulted and local highways have been built for which there is no economic justification. Some townships are covered with a network of concrete roads, and in some cases a considerable portion of these highways have had street lighting installed. The result is that in this area the charges now falling due to meet payments on Covert road bonds are greater than the property owners can bear.

At the present time there are not many new Covert projects, but there are heavy payments remaining on previous work. The report of the legislative committee summarizes the situation well when it states that "This act has undoubtedly caused considerable tax distress throughout certain communities of the State and undoubtedly it should be amended. In fact, there are many persons who advocate its repeal. However, this action does not seem necessary, as it is believed that it can be amended in such a way as to preserve its good features and eliminate its bad features. Improvements which can be constructed only through the medium of this law are essential to the welfare of the people in many sections of the State and until a satisfactory substitute, which provides such facilities under a sounder plan of financing, is proposed, the revision of the present law rather than its repeal seems more logical."

The drainage law is very similar to the Covert law. The number of projects undertaken under this law greatly exceed the Covert projects, but the average expenditure is much less.

In 1930 there were over $70,000,000 of outstanding bonds covering these local highway and drainage undertakings; of this total, $54,000,000 were highway bonds and $16,000,000 drainage bonds. Thus, in Michigan, both in urban and rural areas, special assessments are responsible for a large portion of the imposts. Each special assessment project is handled separately and not as a part of the general business of the community. In many counties records can be obtained only with difficulty, if at all. Although a very large portion of public improvements have been financed through the special assessments, present economic conditions have served to check the Covert and drainage projects in Michigan.

INDEBTEDNESS

The outstanding debt of the State and its subdivisions in 1930 was $816,388,500. The sum of $401,567,700, or almost one half of the total, was incurred for public

[5] Report of the State commission of inquiry into taxation, 1930. Pp. 90 and 91.

benefits such as the protection of person and property, sanitation, and parks. Highways and education were almost equally responsible for the remainder, $202,613,000, or 24.8 percent, being incurred for highways and $204,307,500, or 25 percent, being incurred for schools. For general governmental purposes the indebtedness was less than 1 percent. This debt was incurred primarily for public buildings, such as courthouses and city halls.

All of the governmental units of Michigan may incur indebtedness. Of the State total the greatest portion is within the municipalities, which had a total indebtedness of $633,133,000. Townships had an indebtedness of $64,058,000. The county debt was $35,710,300, and that of the State $83,487,200. Of the State debt, $30,000,000 were obligations incurred for the payment of the soldiers' bonus and $50,000,000 for highway purposes. Of the county and township debt, the total Covert road bonds have been apportioned between the counties and townships in the ratio of the respective obligations assumed by each for the retirement.

County debts are not proportionately high. There were only six counties having obligations of over $1,000,000. Outside of Wayne County, practically all of the county debt was for highways.

The greatest amount of indebtedness is found in the cities. In the small places, indebtedness was $77.32 per capita and in the city of Detroit $239.44. There are individual communities in the Detroit area where the per capita indebtedness is greatly in excess of this figure.

General indebtedness is limited by statutory limitations. This has resulted in some cases of excessive valuations of property to increase the base for borrowings. Special assessment debts are subject to no such limitations.

The policy of Michigan with respect to debt retirement makes impracticable a table showing the debt service for the year comparable to the tables in the Wisconsin and Illinois reports. Debt service consists of the interest paid and the debt principal retired each year. Total interest payments during 1930 were $36,928,200, of which $4,023,700 was paid by the State government, $20,329,900 by Wayne County, and $12,574,600 by all other units.

A common public policy in Michigan is to issue long-term obligations in the form of sinking-fund bonds. Funds designated for the eventual retirements of these bonds when they become due are paid into the sinking fund. Large amounts of this nature were set aside by the State government during the year, but no obligations were actually paid. Since none of the obligations were retired and the full amount was still outstanding, there was no actual principal retirement.

The State highway bond sinking fund may be cited as an example. During the calendar year 1930 the State paid $2,507,800 interest upon its highway debt. There was paid into the highway bond sinking fund the fiscal year ending 1930, $4,832,900, and in the fisc year ending in 1931, $7,377,900. It is obvious, ther fore, that funds in excess of interest requirements a being set aside annually. Not less than $3,000,000 the gasoline tax raised must be used for the payment interest and principal on State highway bonds, and fro the motor-vehicle licenses an additional amount for tl same purpose of $1,200,000. Therefore, even thoug no bonds are retired, of the motor-vehicle imposts eac year approximately $1,700,000 at the minimum are s aside for bond retirement. In recent years sever millions have been advanced from the highway bon sinking fund to the project for the relocation of th Grand Trunk Railroad right of way, such loans to b repaid with interest in accordance with the statute Substantial amounts of the outstanding State highwa bonds have also been purchased out of the sinking fun and will be held in the fund until time for their retire ment. The same general situation applies to Stat sinking funds for othe bonds and to the indebtec ness of many communitie in Michigan. Thus, ther are large sums set asid and dedicated to retire ment, but no actual can cellation of the obligation which can technically b listed under debt servic

In some cases there i an apparent material re duction in indebtednes when actually nothing c the kind has transpired Enormous obligations i the city of Detroit wer paid, but were replace by others exceeding thos which were retired. A tabulation, therefore showing the principal pai on debt would indicat that the State governmen was retiring no indebted ness, while actually larg amounts were being se aside for that eventual use, and that the city of Detroi was making a material reduction in its obligations, whe actually new debts were being incurred of the sam nature and to a greater extent than the amounts retired

MICHIGAN EXPENDITURES IN 1930

The total expenditures (exclusive of principal payments on bonds and loans) by all units of government in Michigan in 1930 were $417,899,500, of which $105,177,100, or 25.2 percent, was expended for highways and streets; $131,495,300, or 31.5 percent, for education; $164,966,900, or 39.4 percent, for public benefit; and $16,260,200, or 3.9 percent, for other governmental purposes.

Expenditures by the State were $69,141,200, or 16.5 percent; by the counties, $68,910,300, or 16.5 percent; by Detroit, $136,116,600, or 32.6 percent; by other incorporated places, $108,417,100, or 25.9 percent; by the townships, $35,314,300, or 8.5 percent.

Of the total expenditures, 24.3 percent was made in rural areas (townships); 5.4 percent in incorporated places having a population of 2,500 or less; 9.9 percent in places of 2,500 to 15,000 population; 12.2 percent in places of 15,000 to 75,000 population; 7.3 percent in places of 75,000 to 400,000 population; and 40.9 percent in Detroit.

GENERAL EXPENDITURES ANALYZED

In Michigan in 1930, $384,732,300 was the public charge levied against persons and property within the State. For the same year the public expenditures excluding principal payments on bonds and payment into sinking funds, amounted to $417,899,500. There can be no exact balance between receipts and expenditures. Taxes are imposed for two purposes. The first is to defray certain current public costs. The second is to pay public debts falling due which were incurred to finance public operations in prior years. A further difference in addition to the time element is that the imposts shown are the taxes as levied. It is possible that in some levies comparatively large sums may remain delinquent in any one particular year although eventually practically the entire amounts will be collected.

The funds for expenditures come from current taxes and from borrowings both in the form of temporary

loans and long-time obligations. There are also frequent surpluses or balances in accounts which may be used for current undertakings.

Thus generally the funds from taxes are used to meet current and past public costs; the funds for current expenditures are derived from the concurrent taxes and from loans to be repaid in the future. In general, however, public financial transactions tend to change gradually, and there is not a great difference between the total taxes imposed for a given year and the total expenditures within the same year. This is particularly true of the less complex units of government.

Of the total of public expenditures in Michigan of $417,899,500, $105,177,100, or 25.2 percent of the total, was expended for all roads and streets.

Included in this total are the costs necessary for keeping all public highways in both cities and rural communities open for travel. Highway expenditures are explained in great detail in the section following the discussion of expenditures in general.

The sum of $131,495,300, or 31.5 percent of the total, was expended for education. This includes the common school system, institutions of higher learning, and libraries.

The sum of $164,966,900, or 39.4 percent of the total, was expended for general public benefits. Included in this classification are all costs pertaining to protection of person and property, public health, recreation, and such items.

For other governmental purposes, the expenditure was $16,260,200, or 3.9 percent of the total. Under the cost of government are included only the executive and legislative departments. Officials such as sheriffs, State's attorneys, and others engaged in the enforcement of law and order are classified under the public benefit section.

TABLE 11.—*All State and local expenditures in 1930 by units of government*

Expended by—	Total amount	Percent	Per capita				
			Highways	Education	Public benefit	Government	Total
State	$69,141,200	16.5	$6.86	$2.12	$4.90	$0.40	$14.28
Counties	68,910,300	16.5	5.82	.13	7.68	.60	14.23
Townships	35,314,300	8.5	5.02	20.01	.96	2.35	28.34
Places up to 2,500	15,581,300	3.7	5.87	33.84	9.11	4.12	52.94
Places 2,500 to 15,000	30,146,100	7.2	7.41	26.92	14.04	3.12	51.49
Places 15,000 to 75,000	38,842,000	9.3	5.02	30.52	20.69	2.30	58.53
Places 75,000 to 400,000	23,847,700	5.7	3.74	26.71	17.30	1.50	49.25
Detroit	136,116,600	32.6	16.76	23.44	44.50	2.07	86.77
Total	417,899,500	100.0	21.72	27.15	34.07	3.36	86.30

Table 11 subdivides the total of all the expenditures within the State of Michigan by the expending agency and gives in addition to the total in each governmental unit the amount per capita expended for every purpose.

Table 12 summarizes in terms of percentages many of the important relationships disclosed by the study. A careful examination of this table will disclose many interesting facts and tendencies in Michigan public expenditures. The first major division of the table shows, in percentage form, the expenditures made by each unit of government for each purpose. The State and county expenditures were primarily for highways and public benefits. The expenditures of the municipalities and townships were primarily for public benefits and education. The expenditures for public benefit are greatest in the areas of dense population. General government costs exerted but a small influence upon the total, being responsible for but 3.9 percent of all expenditures.

The second division of the table subdivides by governmental units the total expenditure for each purpose. It can be seen that of the total highway expenditures the major portions were made by the State, the counties, and the city of Detroit.

Just as all taxes are finally paid by the townships and urban communities of the State, so also the expenditures by the State and counties are made either in or for these same subdivisions. Table 13 gives the per capita total expenditures in the various units of government made both by their governing bodies and in or for them by the State and counties.

TABLE 13.—*Per capita expenditures by purpose in the several classes of place*

Expended in—	Highways	Education	Public benefit	Government	Total
Townships	$43.19	$22.14	$12.92	$3.29	$81.54
Places up to 2,500	14.60	36.15	20.84	5.22	76.81
Places 2,500 to 15,000	10.43	29.20	26.63	4.20	70.46
Places 15,000 to 75,000	6.73	32.75	33.98	3.23	76.69
Places 75,000 to 400,000	5.65	28.89	26.54	2.22	63.30
Detroit	21.52	25.66	58.70	3.06	108.94
Entire State	21.72	27.15	34.07	3.36	86.30

Table 13, allocating the total expenditures among units of government, is comparable with table 3, allocating total taxes in the same general manner. Subject to the difference previously noted as to the base for current tax levies and current expenditures, it is possible to compare directly the taxes paid by the units of government and the expenditures made within them. These facts are shown in table 14.

TABLE 12.—*Percentage relationships of expenditures by purpose and by units of government in 1930*

Unit of government	Percentages of expenditures of the units of government for the purpose indicated					Percentage of the total expenditures for each purpose made by each unit of government				
	Highways	Education	Public benefit	Government	All purposes	Highways	Education	Public benefit	Government	All purposes
State	48.1	14.8	34.3	2.8	100.0	31.6	7.8	14.4	11.8	16.5
County	40.9	.9	54.0	4.2	100.0	26.8	.5	22.6	17.8	16.5
Townships	17.7	70.6	3.4	8.3	100.0	6.0	19.0	.7	18.0	8.5
Places up to 2,500	11.1	63.9	17.2	7.8	100.0	1.6	7.6	1.6	7.4	3.7
Places 2,500 to 15,000	14.4	52.3	27.3	6.0	100.0	4.1	12.0	5.0	11.2	7.2
Places 15,000 to 75,000	8.6	52.1	35.4	3.9	100.0	3.2	15.4	8.3	9.4	9.3
Places 75,000 to 400,000	7.6	54.2	35.1	3.1	100.0	1.7	9.8	5.1	4.5	5.7
Detroit	19.3	27.0	51.3	2.4	100.0	25.0	27.9	42.3	19.9	32.6
Total	25.2	31.5	39.4	3.9	100.0	100.0	100.0	100.0	100.0	100.0

TABLE 14.—*Per capita valuation, taxation, and expenditures in 1930*

Class of place	Valuation	Taxation	Expenditures	Ratio of per capita expenditures to per capita taxation
Townships	$1,424	$55.97	$81.54	1.46
Places 2,500 or less	1,116	63.34	76.81	1.21
Places 2,500 to 15,000	1,333	77.46	70.46	.91
Places 15,000 to 75,000	2,010	93.96	76.69	.82
Places 75,000 to 400,000	1,768	82.42	63.30	.77
Detroit	2,151	94.82	108.94	1.15
Average for State	1,744	79.45	86.30	1.09

COMPARISON OF TAXES AND EXPENDITURES SHOWS SHIFT OF FUNDS FROM URBAN TO RURAL AREAS

The ratio of the per capita expenditures to the per capita taxation is some index of the flow of funds to the less populous areas from other communities. For example, the expenditures within the townships were 46 percent greater than all taxes paid by them. This does not necessarily mean that the townships are benefited to this extent. When direct aids are paid to local authorities and are expended by them, such as is the case with certain school funds, there is a distinct benefit. On the other hand, the construction of a through highway between two large cities may be of more benefit to the residents of the urban communities than to those of the rural districts through which it passes. The ratios also are in some cases materially affected by expenditures financed from borrowings. This causes the difference between taxes paid and expenditures in certain urban communities to be less than would otherwise be the case.

A somewhat different analysis showing the relationships by percentages rather than by per capita figures also indicates this general shift in funds. (See table 15.) In connection with the financial data are given the percentages of population and vehicle ownership in the units of government.

TABLE 15.—*Distribution of population, motor-vehicle ownership property valuation, taxes paid, and expenditures made in the several classes of place in 1930*

Class of place	Population	Motor vehicle ownership	Valuation	Taxes paid	Expenditures made
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Townships	25.7	24.6	21.0	18.1	24.3
Places up to 2,500	6.1	7.5	3.9	4.8	5.4
Places 2,500 to 15,000	12.1	12.7	9.3	11.8	9.9
Places 15,000 to 75,000	13.7	13.7	15.8	16.2	12.2
Places 75,000 to 400,000	10.0	11.1	10.1	10.4	7.3
Detroit	32.4	30.4	39.9	38.7	40.9
Total	100.0	100.0	100.0	100.0	100.0

HIGHWAY EXPENDITURES

In dealing with highway and street expenditures, it is important to distinguish an expenditure made under the authority and control of a governmental unit from an expenditure made within the boundaries of the area. For example, under the first concept, a city street expenditure is the amount expended by city authorities; under the second concept, a city street expenditure is the amount expended on all streets within the city, whether by municipal authorities, the county, the State, or other agencies. It is believed that in this summary, in the expenditure tables, this distinction has been made sufficiently clear to avoid difficulty.

Table 16 gives the total amounts expended by units of government upon the highway systems, and also the division as to the area in which the funds were expended.

TABLE 16.—*Highway and street expenditures in 1930*

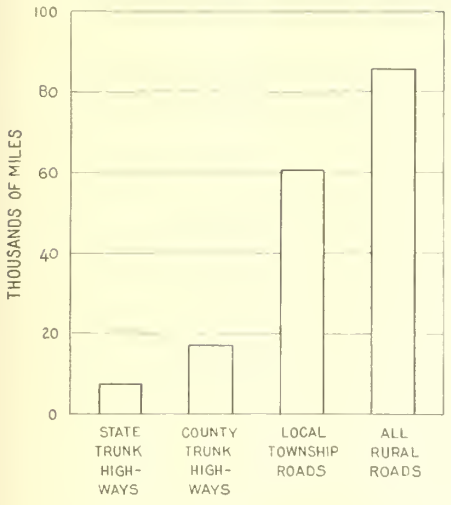
Unit of government	Expended by each unit of government		Expended on—	
	Amount	Percent	Rural roads	Urban streets
State (State system)	$33,225,000	31.6	$27,784,000	$5,441,000
County (county system)	28,185,300	26.8	19,764,100	8,421,200
Townships	6,264,400	6.0	6,264,400	
Places 2,500 or less	1,726,200	1.6		1,726,200
Places 2,500 to 15,000	4,337,200	4.1		4,337,200
Places 15,000 to 75,000	3,330,000	3.2		3,330,000
Places 75,000 to 400,000	1,808,800	1.7		1,808,800
Detroit	26,300,200	25.0		26,300,200
Total	105,177,100	100.0	53,812,500	51,364,600

The concept of highway expenditures in this survey differs from that often found in public accounting. Only those expenditures have been considered as highway expenditures which pertain directly to the construction and maintenance of the roads themselves. Often any expenditure made upon or near a street or under control of the local public works department is considered as a street expenditure. Therefore, often the cost of ornamental lighting systems, sidewalks, sewers, and water systems are shown in city statements as street expenditures. It is also customary to omit from public expenditures any projects financed through special assessments. The highway expenditures as shown in these surveys include all amounts expended for such purposes, whether financed from general levies, special assessments, or specific motor vehicle imposts.

Upon all Michigan roads and streets in 1930 there was expended a total of $105,177,100. The total taxes imposed for street and highway purposes in the same year amounted to $111,853,900, exceeding the expenditures by $6,676,800. Of the total there was expended by the State, $33,225,000, or 31.6 percent; by the counties, $28,185,300, or 26.8 percent. The townships expended $6,264,400, or 6.0 percent, and the cities the remaining $37,502,400, or 35.6 percent. The city of Detroit was responsible for $26,300,200 of all street and highway expenditures made within the year. This is the division of expenditures by expending agencies.

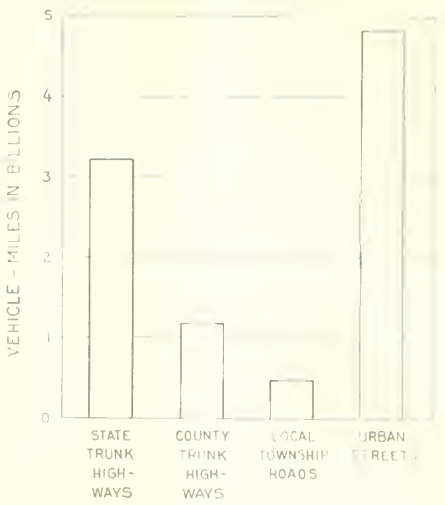
A considerable part of expenditures by the State of Michigan and by the counties is on parts of and for highway systems lying within village and city limits. The State expended $5,441,000 within urban communities, and the counties, $8,421,200. If these amounts are added to the sums expended by the municipal authorities, it will be found that $51,364,600 was expended upon urban streets and $53,812,500 on rural roads.

Table 17 gives the distribution of highway and street expenditures in 1930 according to the governmental agencies providing the funds. There is also a division between those expenditures which were made out of current tax revenues and those which were made out of loan and reserve funds. Federal-aid payments to Michigan in 1930 amounted to $2,934,000. Expenditures out of current taxes (including Federal aid) totaled $85,390,900, or 81.2 percent of the total. The sum of $19,786,200 was expended out of loans and

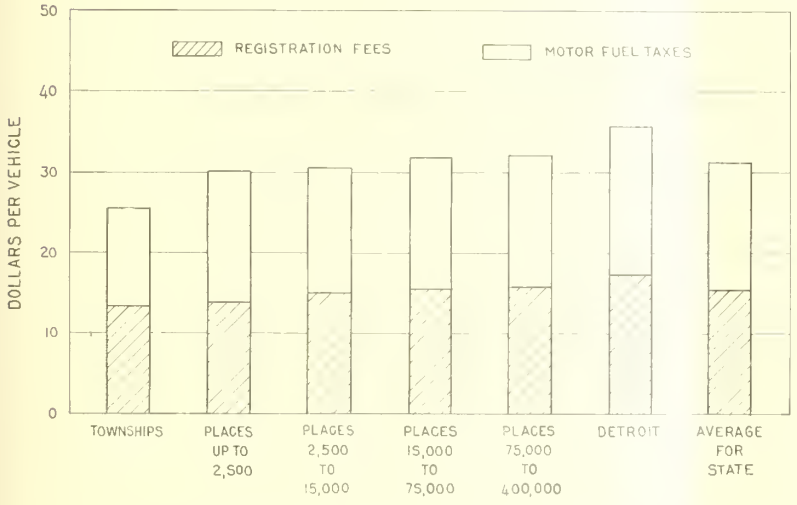


TOTAL MILEAGE OF RURAL HIGHWAY SYSTEMS

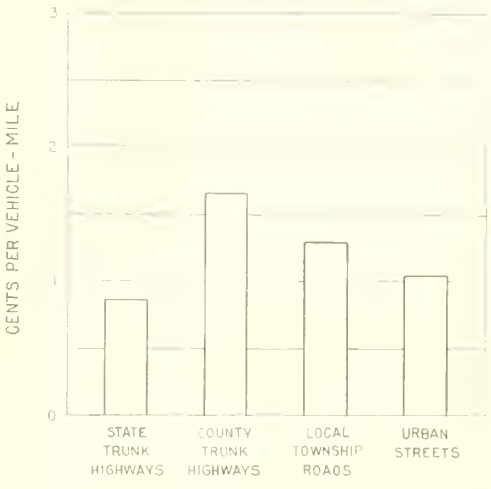
AVERAGE DENSITY OF TRAFFIC ON RURAL HIGHWAY SYSTEMS (FROM MICHIGAN TRANSPORT SURVEY)



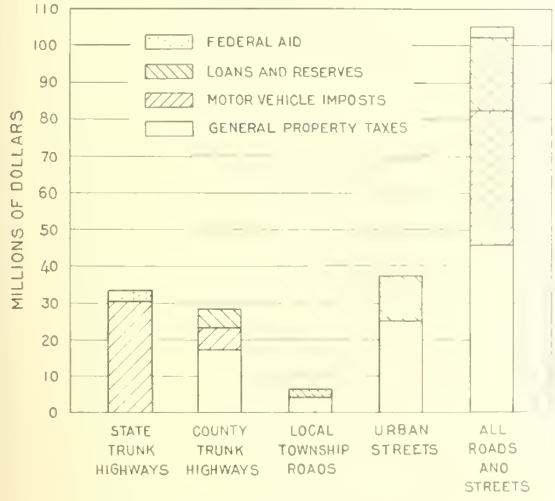
TOTAL TRAFFIC OF MICHIGAN VEHICLES ON RURAL HIGHWAY SYSTEMS AND URBAN STREETS (FROM MICHIGAN TRANSPORT SURVEY)



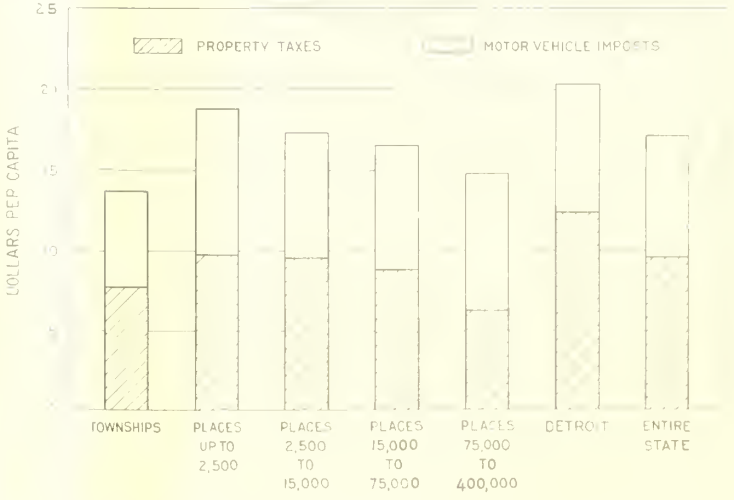
AVERAGE REGISTRATION FEES AND MOTOR FUEL TAX PAYMENTS BY MOTOR VEHICLE OWNERS IN THE SEVERAL CLASSES OF LOCAL UNIT AND THE STATE AS A WHOLE (TABLE 10)



EXPENDITURES PER VEHICLE-MILE ON RURAL HIGHWAY SYSTEMS AND URBAN STREETS (TABLE 9)



TOTAL EXPENDITURES ON HIGHWAY SYSTEMS AND URBAN STREETS, SHOWING SOURCES OF FUNDS (TABLE 22)



PER CAPITA CONTRIBUTIONS BY TAXPAYERS IN THE SEVERAL CLASSES OF LOCAL UNIT TO CURRENT TAXES EXPENDED ON ROADS AND STREETS (BASED ON TABLE 22)

GRAPHICAL SUMMARY OF RELATIONS BETWEEN HIGHWAY TAXES, TRAVEL, AND EXPENDITURES IN MICHIGAN IN 1930.

TABLE 17.—*Funds expended on the several highway systems and the city streets in 1930, and the approximate amounts and percentages of these funds provided by imposts made by the various governmental units, and by loan and reserve funds*

Highway system and form of revenue	Governmental agency providing funds				Total	Percentage of total current tax funds	Percentage of total funds
	Federal government	State	Counties	Local governments			
State trunk system:							
Loans or reserves							
Current taxes	$2,934,000	$30,291,000			$33,225,000	38.9	
Total	2,934,000	30,291,000			33,225,000		31.6
Percent	8.8	91.2			100.0		
County highways:							
Loans or reserves			$5,059,000		$5,059,000		
Current taxes		$6,230,000	16,896,300		23,126,300	27.1	
Total		6,230,000	21,955,300		28,185,300		26.8
Percent		22.1	77.9		100.0		
Local rural (township) roads:							
Loans or reserves				$2,251,300	$2,251,300		
Current taxes				4,013,100	4,013,100	4.7	
Total				6,264,400	6,264,400		6.0
Percent				100.0	100.0		
City streets outside Detroit: [1]							
Loans or reserves				$688,100	$688,100		
Current taxes				10,514,100	10,514,100	12.3	
Total				11,202,200	11,202,200		10.6
Percent				100.0	100.0		
Detroit streets: [1]							
Loans or reserves				$11,787,800	$11,787,800		
Current taxes				14,512,400	14,512,400	17.0	
Total				26,300,200	26,300,200		25.0
Percent				100.0	100.0		
All roads and streets:							
Loans or reserves			$5,059,000	$14,727,200	$19,786,200		
Current taxes	$2,934,000	$36,521,000	16,896,300	29,039,600	85,390,900	100.0	
Total	2,934,000	36,521,000	21,955,300	43,766,800	105,177,100		100.0
Percent	2.8	34.7	20.9	41.6	100.0		

[1] Does not include expenditures by State and counties on streets which are part of the State and county systems.

reserves; of this amount $11,787,000 was indebtedness incurred for streets in Detroit.

Of the funds expended, Federal aid accounted for 2.8 percent; 34.7 percent came from State imposts (motor-vehicle fees and motor-fuel taxes), and the remainder from the taxes levied and indebtedness incurred by the counties and local governments.

The State highway system is an enterprise supported jointly by the United States and the State. The county highway activities are carried on by receipts from both State and county revenues, the present county revenues being about two and one half times the funds raised from the State. These State funds for the county system are given directly as a State aid toward a county program over which the State exercises no jurisdiction except such advisory services as are requested. The other highway systems are under local control alone and are supported by local funds. Under the Covert road act parts of several localities and even counties may be consolidated in a special road district for the construction of a specific highway.

COMPARISONS MADE BETWEEN TRAVEL AND EXPENDITURES

The facts developed by the Michigan transport survey [6] furnish the basis for comparisons between travel and expenditures. So far as the totals as divided between the urban and rural systems are concerned, both travel and expenditures were almost equal in these areas. Fifty-one and two tenths percent of all highway expenditures was made upon rural roads, and these roads carried 50.3 percent of all of the traffic, while city and village streets were responsible for 48.8 percent of the expenditures and carried 49.7 percent of the total traffic.

This rural travel may be further subdivided by highway systems. Of the total expenditure for highways, 26.4 percent was made upon the State trunk system in rural areas (see table 16). These same highways carried 33.1 percent of the total travel. Upon the same basis, 18.8 percent of all highway expenditures was made upon rural county roads which carried 12.2 percent of the total travel. A very small percentage of both expenditures and travel is chargeable to the local township roads. These roads cost 6 percent of the total and carried 5 percent of all of the traffic.

From the facts ascertained by this survey and by the Michigan transport survey, it is possible to express highway expenditures in terms of cents per vehicle-mile of travel. Here again it is necessary to distinguish between different classifications of highway systems.

Certain routes through cities are partly or wholly under county or State jurisdiction with respect to highway construction and maintenance. In most cases expenditures for work on such routes are made directly by county or State, for what may be considered county or State roads. The condition is exactly the same as when a county constructs a county road across an individual township, or when the State builds across a particular county. From this point of view these expenditures are properly chargeable to county or State roads, and not to city streets. This position was taken in the recent report on the Michigan transport survey already referred to, and certain road and street costs per vehicle-mile were ascertained.

It is interesting to note that if, with respect to city streets, all payments by counties and State for work

[6] See PUBLIC ROADS, vol. 13, no. 12, February 1933.

TABLE 18.—*Expenditures per vehicle-mile of travel in 1930–31 on the several highway systems of Michigan, as determined by the Michigan transport survey* [1]

Highway system	Expenditures per vehicle-mile	Annual vehicle-mile on system
	Cents	
Township roads	1.29	484,647,365
County roads [2]	2.37	1,191,399,055
Trunk lines [2]	1.05	3,213,699,440
City streets	.78	4,830,371,675
Average, all highways	1.18	

[1] See Public Roads, vol. 13, no. 12, p. 193.
[2] Entire system, including portions within incorporated areas.

TABLE 19.—*Expenditures per vehicle-mile of travel on Michigan highways in 1930, as determined by the Michigan financial survey*

Highway system	Amount
	Cents
State trunk lines [1]	0.86
County roads [1]	1.66
Township roads	1.29
Urban streets	1.06
Average, all highways	1.08

[1] Does not include portions within incorporated areas.

on routes through cities are classified as city expenditures, the general relations of the unit vehicle-mile costs, though altered, are not so far changed as to produce substantially different conclusions. The expenditure per vehicle-mile for township roads is unchanged. The corresponding expenditure for county roads is reduced, but retains the same relative place. The expenditures for city streets obviously rise and those for State highways are decreased, producing a change which reverses the relative order of these two items but which does not displace either with respect to town or county expenditures. Table 18 gives the expenditures per vehicle-mile as previously published in the report of the Michigan transport survey. Table 19 gives the revised figures based on the alternate method of classifying expenditures. Slight differences occur because the traffic survey was based on the fiscal year and the financial survey on the calendar year.

It should be observed that the values given in tables 18 and 19, based on the actual expenditures of a single year, do not represent the true annual cost of improving and maintaining the several systems of roads and streets. However, highway expenditures do not vary widely from year to year. For this reason it is believed that the 1930 expenditures per vehicle-mile present a fairly accurate picture of the relations between travel and expenditures on the various systems.

RELATIONS ESTABLISHED BETWEEN HIGHWAY TAXES, TRAVEL, AND EXPENDITURES

The total mileage of rural highways in Michigan in 1930 was 85,080. The State highway system consisted of 7,691 miles, of which 4,671 miles were Federal-aid highways. The county highway system contained 17,175 miles, and the local township roads, 60,214 miles. No data are available on mileage of village and city streets.

Expenditures on the State highway system in 1930, including portions within urban areas, were $33,225,000; on the county systems, including portions within urban areas, $28,185,300; on the township local roads, $6,264,400; on Detroit streets, $26,300,200; and on other streets, $11,202,200. Expenditures on all rural roads were $53,812,500; on all urban streets, $51,364,600.

Of the total property taxes expended on all roads and streets, 36.8 percent was expended on county trunk highways, 8.7 percent on local township roads, and 54.5 percent on urban streets.

Of the total motor-vehicle taxes and imposts expended on all roads and streets, 82.9 percent was expended on the State trunk system, and 17.1 percent was expended on the county trunk systems.

Of the total of all taxes and imposts expended on all roads and streets, rural property and motor-vehicle owners paid 20.5 percent, and travel by rural vehicles made up 20.6 percent of the travel on all roads and streets; urban property and motor-vehicle owners paid 79.5 percent, and travel by urban vehicles made up 79.4 percent of the total travel.

Of the total expenditures on all roads and streets, 48.8 percent was expended on urban streets, which carried 49.7 percent of the total traffic in the State; 26.4 percent was expended on rural State trunk highways, which carried 33.1 percent of the traffic; 18.8 percent was expended on rural county roads, which carried 12.2 percent of the traffic; and 6 percent was expended on local township roads, which carried 5 percent of the traffic.

CONSTRUCTION AND MAINTENANCE EXPENDITURES ON STATE SYSTEM SHOWN

The total expenditures by highway systems include construction and maintenance operations and also general engineering costs, overhead, supervision, and interest upon highway obligations. The condition of local records prohibits the division in the counties, townships, and cities of the total into construction and maintenance costs. Such a classification is possible only for the State highway system. Table 20 gives the total State expenditures for construction and maintenance on Federal-aid and other State trunk highways, and the distribution of these expenditures between rural and urban areas.

The construction item is significant. The place of such activities changes from year to year. Although the totals for the system tend to be rather constant from year to year, the amounts expended in any particular place for any short-time period may vary greatly. The State construction expenditures in urban communities in 1930 are proportionately much greater with respect to maintenance costs than in rural areas. In Michigan the State assumes the responsibility for the construction and maintenance of all highways designated as the trunk system, whether these are roads in townships or connecting streets in cities. It either awards $2,000 per mile to those cities which maintain such streets in a manner specified by the highway department, or the depart-

TABLE 20.—*Construction and maintenance expenditures in 1930 on the State trunk-line system, in both rural and urban areas (interest, engineering, supervision, and other undistributed overhead costs not included)*

Highway system	Construction		Maintenance		Construction and maintenance		
	Amount	Percent	Amount	Percent	Amount	Percent	Percentage of total
Federal-aid system:							
Rural	$12,675,300		$3,347,500		$16,022,800		
Urban[1]	1,735,900		310,000		2,045,900		
Total	14,411,200	79.8	3,657,500	20.2	18,068,700	100.0	60.3
Other State trunk highways:							
Rural	6,830,100		2,094,100		8,924,200		
Urban	2,811,000		141,700		2,952,700		
Total	9,641,100	81.2	2,235,800	18.8	11,876,900	100.0	39.7
Entire State system:							
Rural	19,505,400		5,441,600		24,947,000		
Urban	4,546,900		451,700		4,998,600		
Total	24,052,300	80.3	5,893,300	19.7	29,945,600	100.0	100.0

[1] Urban extensions of highways on the Federal-aid system.

ment itself pays for the construction and maintenance of a 20-foot center lane. These expenditures shown as State highway costs in urban localities include all of these payments by the State for the city portion of the State system. The amounts are not duplicated in the city local expenditures.

STREET AND HIGHWAY EXPENDITURES COMPARED WITH TAXES

All the highway expenditures are made within the boundaries of the several classes of place, and all highway funds come from these same places. The relationship between the taxes paid in the various local units of government and the expenditures made on their highway and street systems is shown in table 21.

TABLE 21.—*Comparison of street and highway taxes and expenditures in 1930 by places*

Class of place	Street and highway taxes paid		Street and highway expenditures		Ratio of expenditures to taxes
	Amount	Percent	Amount	Percent	
Townships	$24,333,700	21.8	$53,812,500	51.2	2.21
Places up to 2,500	6,521,200	5.8	4,296,900	4.1	.66
Places 2,500 to 15,000	12,643,700	11.3	6,104,000	5.8	.48
Places 15,000 to 75,000	15,573,400	13.9	4,466,600	4.2	.29
Places 75,000 to 400,000	10,875,000	9.7	2,737,400	2.6	.25
Detroit	41,906,900	37.5	33,759,700	32.1	.81
Total	111,853,900	100.0	105,177,100	100.0	.94

The street and highway taxes exceeded the street and highway expenditures. The reason for this is that these taxes are levied both for current expenditures and to retire indebtedness. The obligations retired during the year were considerably greater than the new indebtedness incurred for highway purposes. New local construction, both in rural and urban areas, was decidedly less in 1930 than in the few years immediately preceding.

The comparison of expenditures with taxes indicates a decided flow of funds from urban communities to rural areas. The relationships are so affected by the policies of the larger metropolitan units in financing their street programs out of current levies or borrowings as to be of little value without considering the influence of these urban areas. Table 21 would seem to indicate that sufficient revenues were raised in Detroit to meet all of the current highway expenditures made within it. This is not the case. Almost $20,000,000 expended on the Detroit street program in 1930 came from borrowings which ultimately must be added to tax levies. Detroit actually contributed $20,783,900 to the State and county highway programs, and there were expended by these agencies within the city, $7,459,500. Thus, the flow of revenues from such metropolitan centers may be greater than a mere tabulation of current taxes and current expenditures would indicate.

TABLE SHOWS CONTRIBUTION OF TOWNSHIPS AND INCORPORATED PLACES TO CURRENT TAXES EXPENDED ON ROADS AND STREETS

The funds derived from taxes levied in a given year are used to defray current expenses, to retire indebtedness, and to establish reserves for future expenditure. Table 22 deals primarily with that portion of the taxes levied in 1930 for streets and highways, totaling $82,456,900, which was used on the current program of 1930. The total is subdivided into imposts on general property (including special assessments) and imposts on motor vehicles (including the motor-fuel tax). The table shows the amounts contributed by taxpayers in each of the classes of place to current expenditures on State, county, and township roads, and urban streets. It also gives the amounts expended on each system out of loans and reserves, and the amount of Federal aid.

The data given in table 22 should not be confused with the information regarding the total street and highway levies given in tables 6 and 7. Thus it is shown in table 7 that taxpayers in the townships were charged with 21.8 percent of all levies for streets and highways in 1930, while table 22 shows that taxpayers in the townships contributed 20.5 percent of the current taxes expended in 1930 on streets and highways.

Of the total current expenditures, 43.7 percent came from property tax levies, 34.7 percent from motor vehicle imposts, and 21.6 percent from loans and reserves and Federal aid. The property taxes expended amounted to approximately 46 million dollars, of which 36.8 percent was expended upon county roads, 8.7 percent on township roads, and 54.5 percent on city streets. Of the motor-vehicle imposts of approximately 36½ million dollars, 82.9 percent was used for State trunk roads and the remainder on county trunk roads. Most of the borrowing was in the urban communities, where 63 percent of all of the new indebtedness was incurred. Included in the general property imposts are special assessments. In Michigan these include not only levies for city streets but also large amounts for rural construction. The distribution in table 22 is based upon highway systems administered by the various authorities, as distinct from the classification by place of expenditure.

The information given in table 22, when analyzed with reference to other data brought out in this report, leads to certain conclusions regarding the relations existing in Michigan in 1930, between the following factors:

1. Taxes paid for highways by taxpayers in the several classes of local government unit;
2. The funds expended on the several highway systems and the local streets;
3. The relative contributions by motor-vehicle owners in the several classes of local unit to the total travel of Michigan vehicles in the State;
4. The relative amounts of travel on the several highway systems and the local streets.

TABLE 22.—*Approximate amounts of the 1930 taxes and imposts expended on the current highway and street program, listed according to highway system, type of tax, and class of local unit in which the tax was paid*

IMPOSTS ON GENERAL PROPERTY

Paid by taxpayers in—	Highway system										
	State trunk		County trunk		Township local		Urban streets		All highways and streets		
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Percentage of total
Townships			$5,812,400	61.3	$3,675,900	38.7			$9,488,300	100.0	20.6
Places 2,500 or less			844,800	29.7	268,900	9.5	$1,726,200	60.8	2,839,900	100.0	6.2
Places 2,500 to 15,000			1,790,900	32.5	68,300	1.2	3,649,100	66.3	5,508,300	100.0	12.0
Places 15,000 to 75,000			2,450,000	42.4			3,330,000	57.6	5,780,000	100.0	12.6
Places 75,000 to 400,000			1,182,700	39.5			1,808,800	60.5	2,991,500	100.0	6.5
Detroit			4,815,500	24.9			14,512,400	75.1	19,327,900	100.0	42.1
Total			16,896,300	36.8	4,013,100	8.7	25,026,500	54.5	45,935,900	100.0	100.0

IMPOSTS ON MOTOR VEHICLES

Townships	$6,149,000	82.9	$1,264,700	17.1					$7,413,700	100.0	20.3
Places 2,500 or less	2,211,200	82.9	454,800	17.1					2,666,000	100.0	7.3
Places 2,500 to 15,000	3,756,100	82.9	772,500	17.1					4,528,600	100.0	12.4
Places 15,000 to 75,000	4,240,700	82.9	872,200	17.1					5,112,900	100.0	14.0
Places 75,000 to 400,000	3,453,200	82.9	710,200	17.1					4,163,400	100.0	11.4
Detroit	10,480,800	82.9	2,155,600	17.1					12,636,400	100.0	34.6
Total	30,291,000	82.9	6,230,000	17.1					36,521,000	100.0	100.0

ALL IMPOSTS AND TAXES

Townships	$6,149,000	36.4	$7,077,100	41.9	$3,675,900	21.7			$16,902,000	100.0	20.5
Places 2,500 or less	2,211,200	40.1	1,299,600	23.6	268,900	4.9	$1,726,200	31.4	5,505,900	100.0	6.7
Places 2,500 to 15,000	3,756,100	37.4	2,563,400	25.5	68,300	0.7	3,649,100	36.4	10,036,900	100.0	12.2
Places 15,000 to 75,000	4,240,700	38.9	3,322,200	30.5			3,330,000	30.6	10,892,900	100.0	13.2
Places 75,000 to 400,000	3,453,200	48.3	1,892,900	26.4			1,808,800	25.3	7,154,900	100.0	8.7
Detroit	10,480,800	32.8	6,971,100	21.8			14,512,400	45.4	31,964,300	100.0	38.7
Total	30,291,000	36.7	23,126,300	28.0	4,013,100	4.9	25,026,500	30.4	82,456,900	100.0	100.0
Federal-aid	2,934,000								2,934,000		
Loans and reserves			5,059,000	25.6	2,251,300	11.4	12,475,900	63.0	19,786,200	100.0	
Grand total	33,225,000	31.6	28,185,300	26.8	6,264,400	6.0	37,502,400	35.6	105,177,100	100.0	

CONCLUSIONS

1. No funds raised from taxes on property were expended on State highways.

2. Rural property paid no tax for urban streets.

3. Of the total tax on rural property expended for highway purposes—

(*a*) 61.3 percent was expended on county trunk highways;

(*b*) 38.7 percent was expended on local township roads.

4. Of the total tax on urban property expended for highway purposes—

(*a*) 30.4 percent was expended on county trunk highways, including portions within urban areas;

(*b*) 0.9 percent was expended on local township roads;

(*c*) 68.7 percent was expended on urban streets.

5. Since, of the total assessed property valuation of $8,447,141,000, 21 percent, or $1,773,958,100, was rural and 79 percent, or $6,673,182,900, was urban—

(*a*) Expenditures from property taxes for all highways were at the following rates per $100 valuation:
Rural—54 cents.
Urban—55 cents.

(*b*) Expenditures from property taxes for county trunk highways were at the following rates per $100 valuation:
Rural—33 cents.
Urban—17 cents.

(*c*) Expenditures from property taxes for local township roads were at the following rates per $100 valuation:
Rural—21 cents.
Urban—0.5 cent.

(*d*) Expenditures from property taxes for urban streets were at the following rates per $100 valuation:
Rural—no tax.
Urban—37.5 cents.

6. Of the total property taxes expended on all roads and streets—

(*a*) 36.8 percent was expended on county trunk highways;

(*b*) 8.7 percent was expended on local township roads;

(*c*) 54.5 percent was expended on urban streets.

7. Of the total motor-vehicle imposts expended on all classes of roads and streets—

(*a*) Rural motor vehicle owners (residents of townships) paid 20.3 percent, and travel by these same rural owners made up 20.6 percent of the total travel on all classes of roads and streets;

(*b*) Urban motor-vehicle owners paid 79.7 percent, and urban vehicles made up 79.4 percent of the total travel.

8. Of the total motor vehicle imposts expended—

(*a*) 82.9 percent was expended on State trunk highways, including portions within urban areas;

TABLE 23.—*Comparison of taxation and expenditures in Michigan in 1930*

$1,000 in taxes			$1,000 in expenditures				
Residents of—	Pay—	In—	Division by purpose	Subdivision by highway system	Source of expenditures	Amount	Percent
Townships	$128.36	General property taxes, $733.53.	Education, $314.66.	State highways, $79.50	Taxes paid in—		
Places 2,500 or less	32.61				Townships	$14.71	18.51
Places 2,500 to 15,000	84.82				Incorporated places	57.77	72.66
Places 15,000 to 75,000	122.14				Federal-aid	7.02	8.83
Places 75,000 to 400,000	76.35				Loans or reserves		
Detroit	289.25						
Townships	23.31	Motor-vehicles taxes, $114.98.	Highways, $251.68	County trunk highways, $67.45.	Taxes paid in—		
Places 2,500 or less	8.35				Townships	16.94	25.11
Places 2,500 to 15,000	14.31				Incorporated places	38.40	56.94
Places 15,000 to 75,000	16.10				Loans or reserves	12.11	17.95
Places 75,000 to 400,000	13.06						
Detroit	39.85			Township roads $14.99	Taxes paid in—		
					Townships	8.79	58.63
					Incorporated places	.81	5.38
			Public benefit, $394.75.		Loans or reserves	5.39	35.94
Townships	29.58	Miscellaneous taxes, $151.49.	Government, $38.91.	Urban streets, $89.74	Taxes paid in—		
Places 2,500 or less	7.49				Townships		
Places 2,500 to 15,000	18.76				Incorporated places	59.89	66.73
Places 15,000 to 75,000	23.85				Loans or reserves	29.85	33.27
Places 75,000 to 400,000	14.31						
Detroit	57.50						

(*b*) 17.1 percent was expended on county trunk highways, including portions within urban areas;

(*c*) None of these funds were expended on urban streets not on the State or county systems.

9. Of all current imposts and taxes expended on all roads and streets—

(*a*) 36.7 percent was expended on State trunk highways;

(*b*) 28.0 percent was expended on county trunk highways;

(*c*) 4.9 percent was expended on local township roads;

(*d*) 30.4 percent was expended on urban streets by municipal authorities.

10. Of all current imposts and taxes expended on all roads and streets—

(*a*) Rural property and motor-vehicle owners paid 20.5 percent, and travel by rural vehicles made up 20.6 percent of the total travel on all classes of roads and streets;

(*b*) City and village property owners paid 79.5 percent, and travel by urban vehicles made up 79.4 percent of the total travel.

11. Of the total expenditures (including Federal aid and expenditures out of loans and reserves) on all roads and streets—

(*a*) 48.8 percent was expended on urban streets (including portions on the State and county systems), and urban streets carried 49.7 percent of the total traffic in the State;

(*b*) 26.4 percent was expended on rural State trunk highways, which carried 33.1 percent of the traffic;

(*c*) 18.8 percent was expended on rural county roads, which carried 12.2 percent of the traffic;

(*d*) 6 percent was expended on local township roads, which carried 5 percent of the traffic.

FINAL COMPARISON OF TAXES AND EXPENDITURES

As a means of summarizing the general taxation and expenditures in Michigan in 1930, table 23 has been prepared. It shows a comparison between the sources of $1,000 in taxes and the manner in which $1,000 was expended, and is based on tables given previously in the report. The comparison, however, is not a direct one, for the $1,000 unit cannot be identical for both taxes and expenditures. If it were desired to make the two sides of the tabulation balance exactly, as in the case of a true balance sheet, it would be necessary to include proceeds from bonds and loans on the left of the double line and principal payments on the right, as well as to make due allowance for balances carried over and carried forward. The complete figures necessary for such a presentation are not available, so that expenditures as given cannot be said to balance receipts from taxes. It is believed, however, that table 23 gives a helpful picture of the relation between tax money received and actual expenditures in the year 1930.

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TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

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▼ ▼ ▼

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

MAY 31, 1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,337.1	5,225,216.68	2,612,608.22	58	132.0	116.6	248.6						3,199,828.75	Alabama
Arizona	1,233.3	2,522,425.72	970,918.63	73	51.5	180.0	231.5						55,626.44	Arizona
Arkansas	1,923.0	4,735,527.91	2,256,876.84	61	132.3	92.8	225.1	901,669.55	450,834.71	80.5	6.4	86.9	805,152.02	Arkansas
California	2,478.2	8,921,645.87	1,695,047.65	75	137.1	70.4	207.5	73,760.41	17,153.65	.8	2.0	2.8	58,863.59	California
Colorado	1,815.9	3,489,391.13	1,585,433.59	71	151.0	32.5	183.5	394,215.58	177,396.98	27.9	.2	28.1	243,852.04	Colorado
Connecticut	294.8	4,563,063.46	1,795,512.12	70	51.0		51.0						127,525.67	Connecticut
Delaware	381.1	755,181.00	150,125.44	63	23.6	15.9	39.5							Delaware
Florida	661.0	6,542,819.34	3,135,268.41	79	170.6		170.6	123,647.45	61,647.96	1.9		1.9	989,164.32	Florida
Georgia	3,189.8	5,048,494.83	2,046,601.46	81	139.3	197.1	336.4						100,373.36	Georgia
Idaho	1,533.5	2,749,053.49	1,012,794.12	79	127.7	130.8	258.5	86,957.69	42,362.30	8.2		8.2	103,290.88	Idaho
Illinois	3,091.6	20,432,379.33	7,831,019.26	76	636.4	60.6	697.0	716,775.42	311,902.26	27.0		27.0	95,967.25	Illinois
Indiana	2,086.6	7,451,152.64	2,978,491.53	86	245.5	20.7	266.2	1,493,884.11	145,628.13	70.6	5.3	75.9	188,054.92	Indiana
Iowa	3,540.2	5,321,273.39	817,331.76	91	261.1	38.5	299.6						29,103.97	Iowa
Kansas	3,997.6	4,133,818.85	1,173,741.72	69	258.1	65.5	323.6	231,480.86	83,452.51	8.6	15.2	23.8	278,982.17	Kansas
Kentucky	1,893.5	4,475,255.11	1,587,617.21	66	187.4	134.3	321.7						188,804.88	Kentucky
Louisiana	1,607.6	6,790,454.94	2,947,282.63	68	54.4	24.8	79.2						34,014.70	Louisiana
Maine	818.9	2,381,995.03	522,942.74	77	70.5	.3	70.8	38,958.62	3,989.09		1.9	1.9	140,840.26	Maine
Maryland	852.6	1,354,165.02	233,695.92	66	53.5	1.5	55.0						25,648.82	Maryland
Massachusetts	876.4	4,115,901.96	972,218.48	61	57.2	4.9	62.1						282,623.33	Massachusetts
Michigan	2,312.5	7,247,532.13	2,837,534.95	65	288.3	115.9	404.2	746,640.00	246,595.00	51.8	.8	52.5	173,006.22	Michigan
Minnesota	4,312.1	4,760,596.74	133,728.91	88	150.9	139.9	290.8						69,532.87	Minnesota
Mississippi	1,812.8	7,168,786.43	3,551,396.46	62	231.1	108.8	339.9	10,801.20	5,400.59	.1	.1	.2	3,063,618.19	Mississippi
Missouri	3,211.9	4,501,301.10	789,999.89	36	175.2	22.9	198.1	686,905.85	127,623.60	27.1	16.6	43.7	8,827.28	Missouri
Montana	2,892.1	6,111,336.57	3,422,721.22	69	466.2	252.9	719.1	89,226.46	50,272.68		23.9	23.9	402,905.27	Montana
Nebraska	4,260.8	6,304,638.44	2,919,138.56	70	165.0	160.3	325.3						40,926.05	Nebraska
Nevada	1,357.3	1,941,156.63	509,173.57	88	39.7	121.2	160.9						117,434.07	Nevada
New Hampshire	433.2	1,066,374.35	434,521.61	75	24.4	3.4	27.8	37,276.50	17,006.65		.7	.7	81,998.32	New Hampshire
New Jersey	632.5	6,084,562.24	2,073,149.85	80	61.0		61.0						109,558.29	New Jersey
New Mexico	2,221.4	3,229,921.75	1,288,681.06	84	172.3	138.8	311.1						149,118.85	New Mexico
New York	3,505.3	18,828,180.05	5,874,630.00	48	490.5	32.0	522.5						351,782.16	New York
North Carolina	2,312.8	5,066,882.02	2,531,836.05	64	551.3	28.1	579.4	297,959.10	148,873.03	28.2	.3	28.5	1,149,990.46	North Carolina
North Dakota	5,219.8	4,330,373.55	1,752,517.19	56	339.4	485.6	825.0	622,682.51	217,198.50	57.8	150.3	208.1	437,293.49	North Dakota
Ohio	3,016.2	9,112,723.09	2,533,346.63	71	204.7	66.8	271.5	76,364.00	23,361.08	1.4	3.2	4.6	188,823.86	Ohio
Oklahoma	2,489.4	4,124,760.46	1,231,752.30	75	194.9	83.8	278.7	528,539.65	105,707.88	38.3		38.3	427,408.94	Oklahoma
Oregon	1,630.6	3,872,289.99	1,551,322.89	66	107.0	73.7	180.7	59,014.89	25,947.79	1.5	1.6	3.1	156,013.79	Oregon
Pennsylvania	3,248.1	10,898,742.73	2,871,818.57	62	377.4	14.1	391.5						51,166.98	Pennsylvania
Rhode Island	271.6	1,071,153.47	346,631.18	47	22.5	4.5	27.0						55,315.03	Rhode Island
South Carolina	1,962.7	3,416,024.48	1,278,467.73	75	161.3	154.7	316.0						11,618.92	South Carolina
South Dakota	4,315.3	3,357,901.41	1,325,224.06	76	235.5	206.9	442.4	88,832.77	43,627.89	8.1	7.8	15.9	251,730.44	South Dakota
Tennessee	1,700.1	5,156,858.77	2,577,708.42	60	163.8	60.1	223.9						576,790.79	Tennessee
Texas	8,017.4	17,762,275.92	5,916,320.86	69	644.9	527.5	1,172.4	759,702.63	235,151.52	58.6	5.3	64.0	81,988.13	Texas
Utah	1,263.0	1,867,108.37	722,209.93	76	122.0	70.9	192.9	111,015.81	33,304.71	2.5	2.6	5.1	162,627.01	Utah
Vermont	392.9	414,151.77	50,299.46	75	19.6		19.6						43,151.97	Vermont
Virginia	1,931.1	4,987,343.35	2,296,272.88	73	231.0	61.6	292.6	81,689.07	40,066.51	4.6		4.6	182,699.06	Virginia
Washington	1,326.7	3,105,552.19	946,420.63	67	113.6	9.7	123.3						59,158.74	Washington
West Virginia	925.6	3,124,589.15	1,273,430.11	75	113.8	8.2	122.0						48,115.11	West Virginia
Wisconsin	2,736.8	6,053,999.53	1,156,521.70	75	148.3	113.0	261.3	52,061.50	21,500.00		.9	.9	265,183.36	Wisconsin
Wyoming	2,124.5	2,871,949.06	917,169.85	86	260.2	147.0	407.2	60,671.62	6,067.16	1.6	10.1	11.7	151,122.56	Wyoming
Hawaii	104.5	1,887,509.64	1,230,021.72	52	41.9		41.9						500,079.00	Hawaii
TOTALS	106,553.7	260,735,791.08	92,669,495.97	69	9,257.9	4,399.5	13,657.4	8,370,733.25	2,642,074.18	507.1	255.2	762.3	16,316,702.58	TOTALS
Construction completed		167,761,000	63,735,000											
Balance uncompleted		92,975,000	28,934,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 5 ▽ JULY 1933

EXPOSURE CABINET FOR TESTING EFFICIENCY OF COTTON MATS IN PROTECTING CONCRETE WHILE CURING

For sale by the Superintendent of Documents, Washington, D.C. - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 5 July 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

COTTON MATS FOR CURING CONCRETE

REPORT OF TESTS TO DETERMINE THE EFFICIENCY OF COTTON MATS OF VARIOUS THICKNESSES FOR THE PROTECTION OF CONCRETE WHILE CURING

Prepared by the Division of Tests, United States Bureau of Public Roads

THE IMPORTANCE of minimizing temperature changes within concrete, especially during the early curing period, is well recognized. In connection with this problem the possibility of utilizing the heat insulating properties of mats made up of several plies of raw cotton separated by loosely woven cotton cloth has been suggested and samples of cotton mats containing 3, 6, and 9 plies of cotton were submitted for test purposes by the Texas State Highway Department. These mats measured approximately 1, 2, and 3 inches in thickness, respectively. Figure 1 gives a general idea of the construction of the mats and also shows the number of plies of cotton in each mat as received. The dry mats showed the following average weights per square foot:

	Pounds
3 ply	0. 7
6 ply	1. 2
9 ply	1. 7

Tests were conducted by the Bureau at the Arlington Experimental Farm to determine the efficiency of these mats for curing concrete. These tests involved two separate investigations, one a study of the heat insulating value of the mats when applied to a concrete surface and the other the ability of the mats to retain moisture within the concrete and thus promote efficient curing.

TEMPERATURE TESTS

First series.—In order to determine the relative protection from heat absorbed from the sun's rays afforded to concrete pavement slabs by the cotton mats, tests were conducted in which the surface temperatures of small concrete slabs covered with these and other materials and exposed to the rays of the sun were measured over the daily temperature cycle.

Six concrete slabs each 2 feet square and 6 inches thick were constructed for the tests, a typical paving mixture being used. In the upper face of each slab, at the midpoint and one fourth inch below the surface, a copper-constantan thermocouple was installed to indicate the surface temperature of the concrete during the tests. Three of these slabs were covered with sections of the three thicknesses of cotton mats, while for comparison the other three were covered as follows: No cover, double burlap, and a bituminous cutback coating.

To produce a greater temperature range in the air surrounding the specimens than would be possible out of doors at the time of year at which the tests were made, thus simulating summer conditions, an insulated box with a glass cover, similar to a horticultural coldframe, was constructed. The walls of the box were 6 inches thick and filled with sawdust. The box was inclined toward the sun at an angle such that the sun's rays passing through the glass cover fell normally upon the surface of the specimens. The appearance of this box and the arrangement of the specimens in it are shown in the cover picture of this issue and in figures 2 and 3.

The first observations were made on March 28, 1933, the slabs being 2 weeks old. The evening before the observations were begun the double burlap covering and the three cotton mats were removed from the frame, laid on the ground, and sprinkled with a hose for 5 minutes. The burlap became saturated at once; but the natural oils in the cotton fiber made the mats very water-repellent, and apparently only a small amount of moisture was absorbed during the prolonged sprinkling. The mats were placed on the concrete with the wetted side up, and the box closed up immediately. Temperature readings were begun before daylight the next morning and were continued throughout the day until after dark. These observations included the temperature in the concrete of each of the

COTTON MATS EFFECTIVE IN HEAT INSULATION AND CURING OF CONCRETE

Experimental determination of the efficiency of cotton mats as a substitute for wet burlap in the curing of concrete reveals the following facts:

1. Cotton mats were proved effective in providing insulation from the heat of the sun's rays. Under the conditions of the tests, the 24-hour temperature range under cotton mats was approximately 35 percent of that which occurred in uncovered concrete and about 60 percent of that in concrete covered with wet burlap.

2. The insulating properties of various thicknesses of mat (ranging from 1-ply, about one third inch thick, to 9-ply, about 3 inches thick) appeared to be about the same.

3. Modulus of rupture tests showed that cotton mats, wet once and placed with the wetted side on the concrete, were as effective in curing as was double-thickness burlap kept continuously wet, the curing period in both cases being 3 days.

4. Cotton mats applied dry proved less effective in curing than wet mats or wet burlap, the specimens developing about 88 percent of the strength of those cured by the other two methods.

5. The various thicknesses of mat used appeared to be equally effective as curing agents for concrete.

six slabs, the air temperature within the box as indicated by a shaded thermocouple, and the air temperature outside the box, also measured in the shade. All of the data obtained in this first series of observations are shown in figure 4, while in figure 5 the ranges from maximum to minimum temperature for the air within the box, the bare concrete, and the slabs covered with the double burlap and the three thicknesses of cotton mat are given.

Following this test all covering materials were thoroughly dried for several days, and on April 3, 1933, the tests were repeated with all materials assembled in a dry condition. The data from these observations are shown in figure 6 and figure 7.

It will be noted in an examination of the data that the cotton mats were very effective in reducing the temperature range to which the concrete under them was subjected and that the difference in effect between the three thicknesses used was small. The temperature range under the cotton mats was approximately 35 percent of that of the bare concrete and about 60 percent of that of the concrete covered with a double layer of burlap.

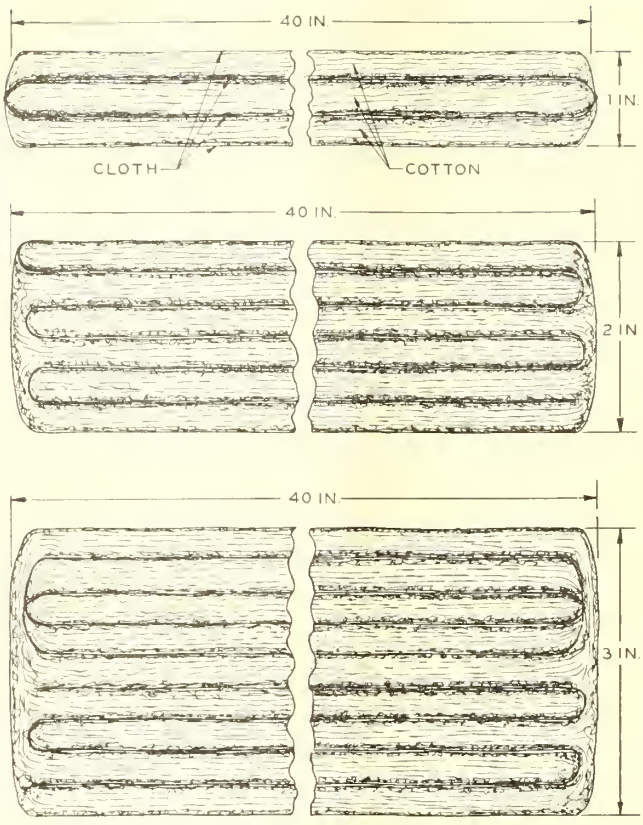

FIGURE 1.—STRUCTURE OF COTTON MATS.

The possibility of making use of the cotton mats for the protection against freezing of concrete exposed to sudden drops in temperature is recognized, and a series of laboratory tests is being initiated to investigate their value for this purpose.

The heat-absorbent properties of the black surface obtained with the bituminous coating is evident in these data. It will be noted that there is relatively little difference between the temperature of the bare concrete and that of the slab coated with bitumen, considerably less than has been found in other previous tests. This was thought to be due to the fact that the concrete was unusually dark in color because the cement employed was a very dark blue-gray, in fact almost black, and raised an interesting

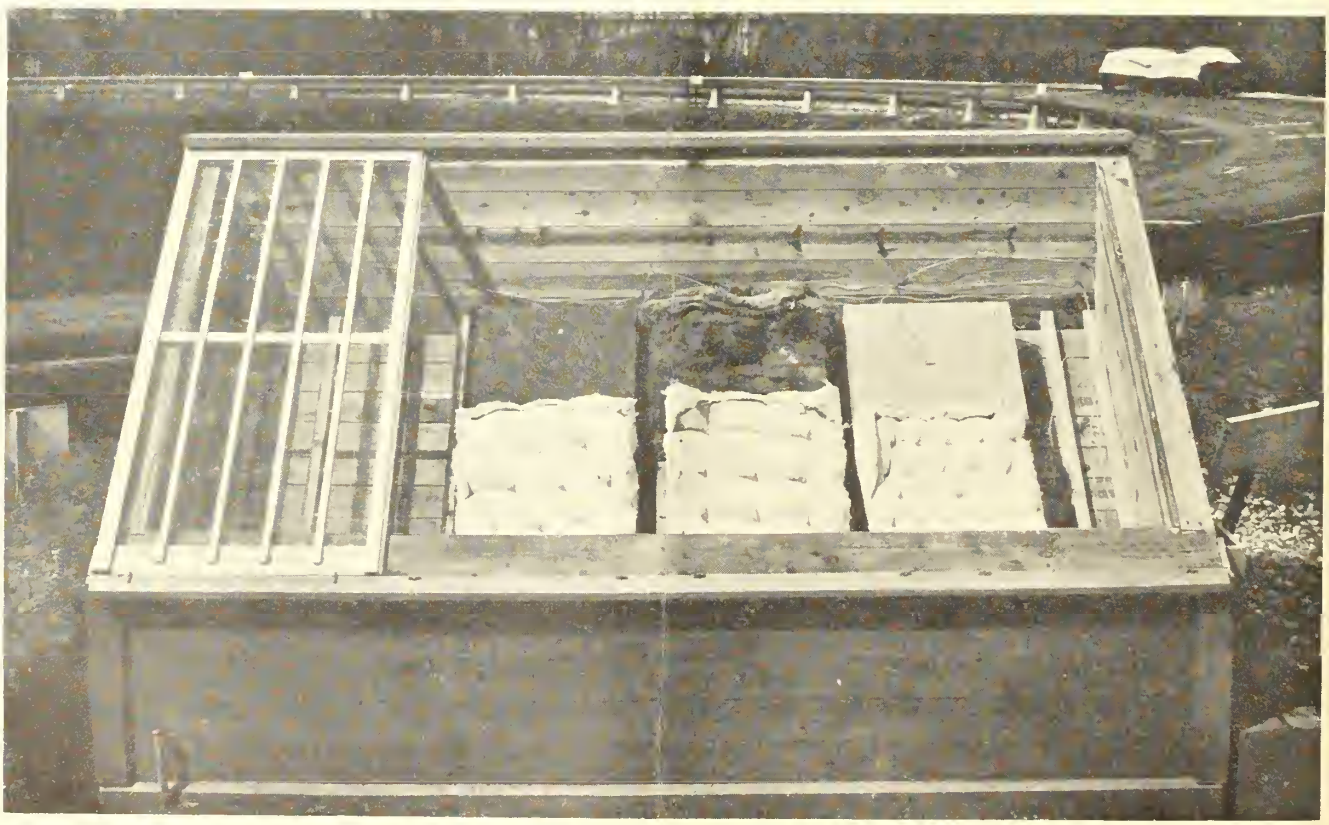

FIGURE 2.—EXPOSURE CABINET OPENED TO SHOW ARRANGEMENT OF TEST SLABS.

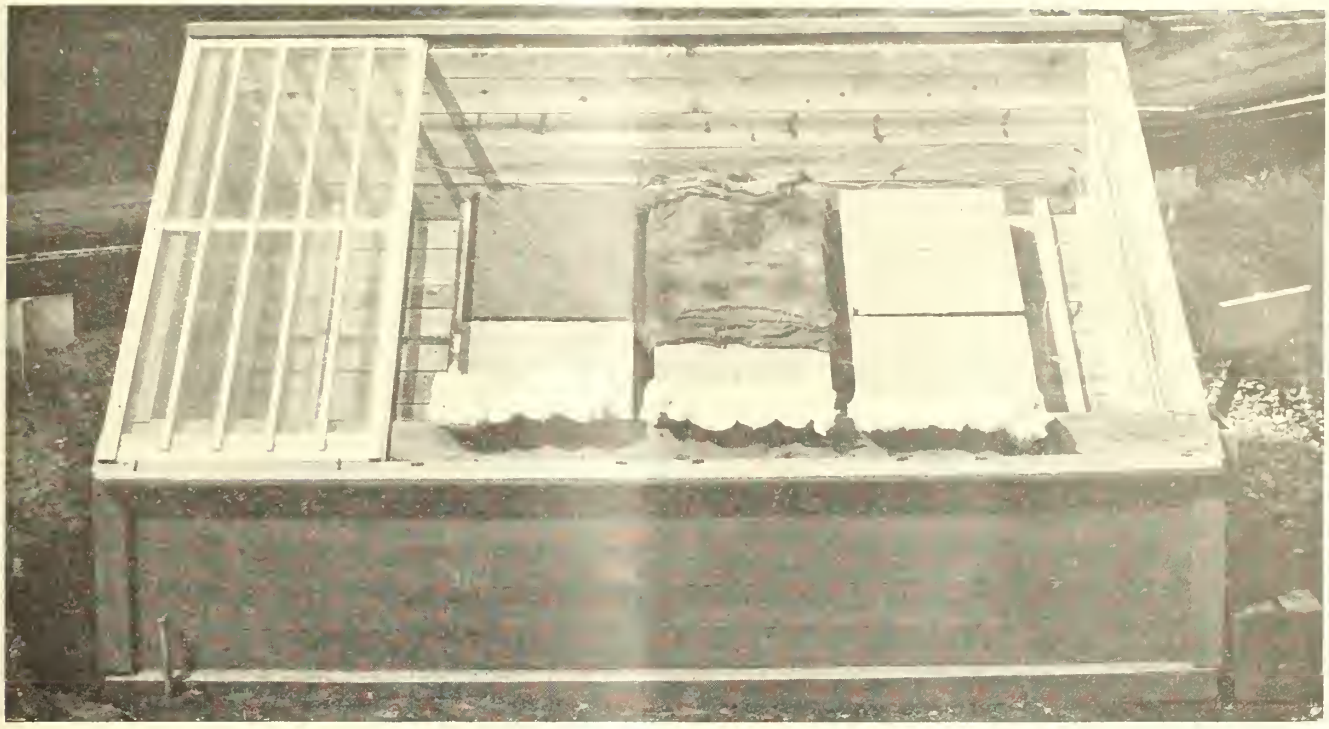

FIGURE 3.—COTTON MATS LIFTED TO SHOW RELATIVE THICKNESS OF EACH.

question as to the effect of cement color on the heat absorption of concrete pavements. Some data on this point appear later in the report.

Taking the mean temperature of the bare concrete slab as a base and comparing with it the mean temperature of the burlap-covered slab and the mean temperature of the group of three slabs which were covered with cotton mats, we find the following relations:

Covering	Decrease in temperature below that of bare concrete
	°F.
Double burlap (damp)	12
Cotton mats (damp)	22
Double burlap (dry)	7
Cotton mats (dry)	19

This comparison gives a good idea of the relative protection from the sun's rays afforded by the burlap and the cotton mats, brings out the temperature-lowering effect of moisture evaporation, and shows that even when thoroughly dry the cotton mat provides a very definite thermal protection for the slab.

The fact that there was so little difference in the temperature of the slabs under the three thicknesses of cotton suggested that mats having less than three plies of cotton might be effective.

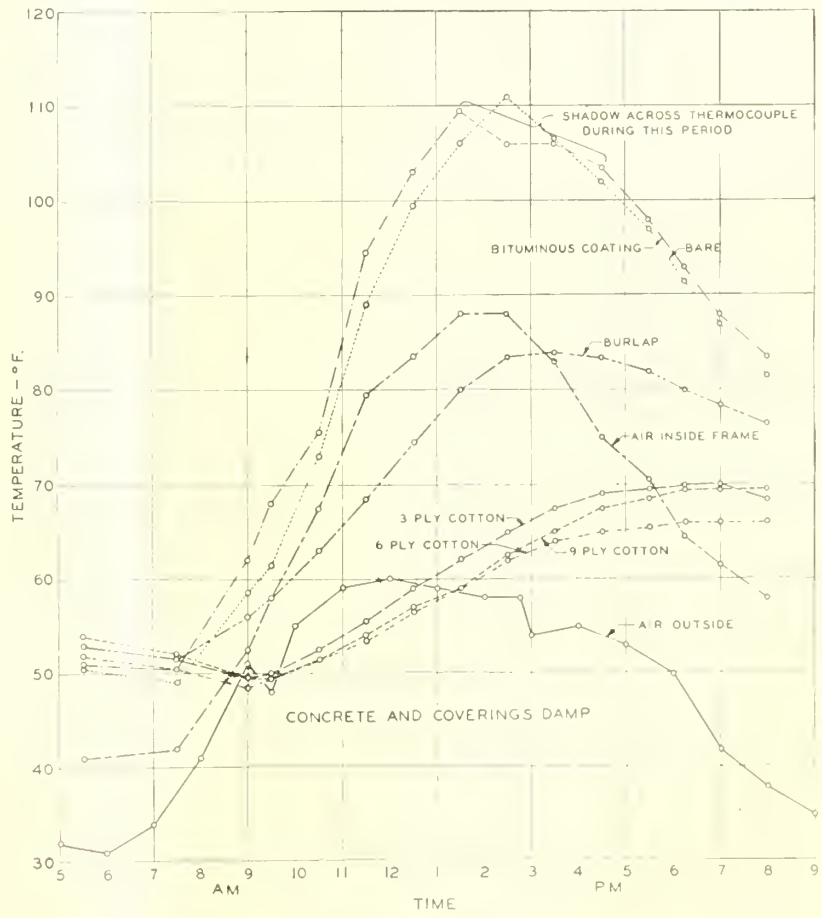

FIGURE 4.—EFFECT OF VARIOUS SURFACE COVERINGS ON THE TEMPERATURE OF CONCRETE EXPOSED TO THE SUN'S RAYS; TESTS MADE MARCH 28, 1933.

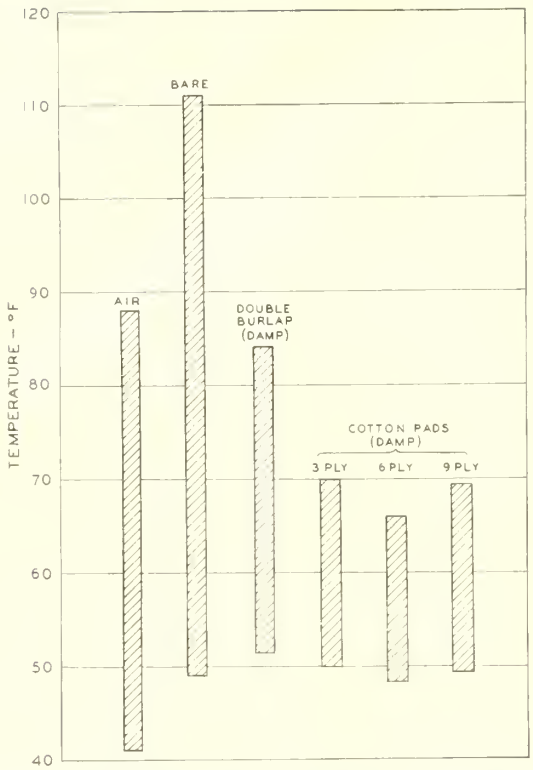

FIGURE 5.—TWENTY-FOUR-HOUR RANGE IN TEMPERATURE OF CONCRETE COVERED WITH VARIOUS MATERIALS AND EXPOSED TO THE SUN'S RAYS; TESTS MADE MARCH 28, 1933.

It is apparent that the exposure cabinet functioned as intended, increasing the air temperature range about 50 percent and creating maximum air temperatures (in the shade) of approximately 90° F., at a time when such temperature conditions could not be even approached out of doors.

Second series.—Because of the opportunity which was afforded to obtain additional data on this important subject and to answer certain questions which had been raised by the first series of observations, additional temperature tests were made. The procedure followed was essentially the same as that previously used, and the points investigated were:

(*a*) The effect of placing the wetted side of the cotton mats down, in contact with the concrete, instead of up.

(*b*) The effect of using fewer plies of cotton in the mats.

(*c*) The effect of the color of the cement on the temperature of the concrete.

(*d*) The effect of certain other coverings.

The first series of temperature tests were made with the wetted side of the cotton mats up, as would be the case if the mats were placed on the road slab and then sprinkled with a hose. The question arose as to whether the insulating effect of the mats would be different were they wet on the lower surface only. In the tests which were made to determine the answer to this question, the temperatures of slabs covered with 1-, 2-, 3-, 6-, and 9-ply cotton mats with the wetted side both up and down were observed over a complete daily cycle in comparison with the temperature of a bare slab. The 24-hour range for each of these is shown in table 1, expressed as a percentage of the range of the bare concrete slab.

TABLE 1.—*24-hour ranges of temperatures in concrete covered with cotton mats of various thicknesses, and in bare concrete*

Covering	24-hour range	
	Wet side up	Wet side down
	Percent	*Percent*
Bare	[1] 100	[1] 100
1-ply cotton mat	40	38
2-ply cotton mat	30	30
3-ply cotton mat	35	35
6-ply cotton mat	30	37
9-ply cotton mat	32	36

[1] Bare concrete slab dry in both cases.

These data indicate that the position of the wetted surface does not appreciably affect the daily temperature range of the concrete.

Under conditions of low humidity it is probable that, if the upper surface of the mat were wet, evaporation would lower somewhat the mean temperature below the mat. In these tests the exposure cabinet was closed and evaporation was limited, so that it was not to be expected that a difference in mean temperature would

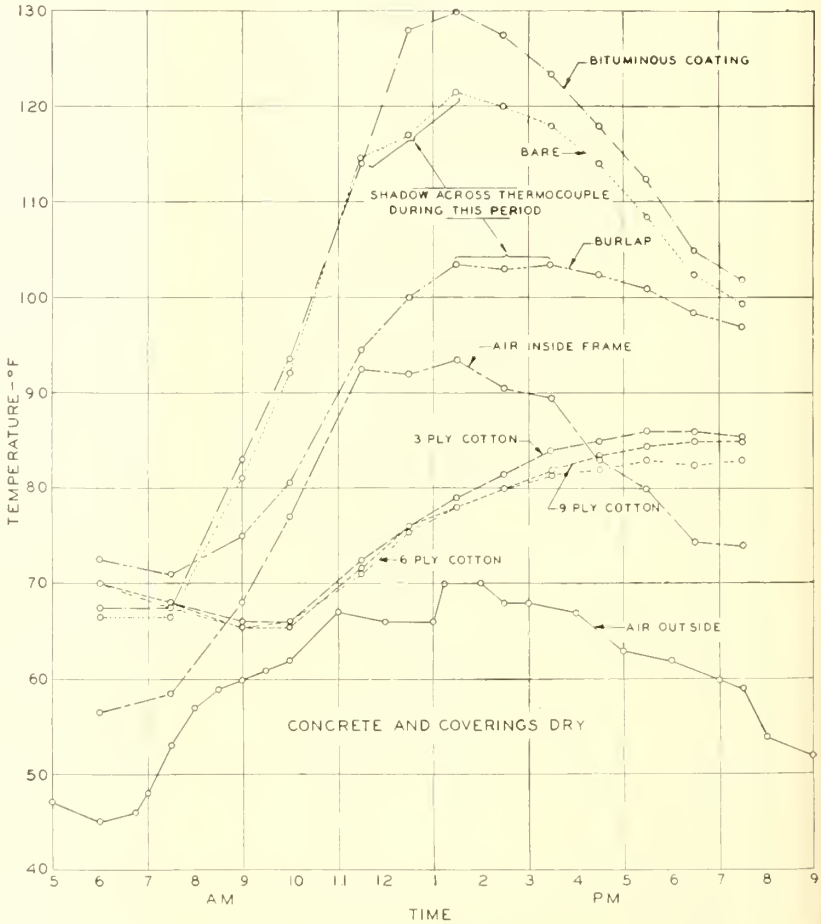

FIGURE 6.—EFFECT OF VARIOUS SURFACE COVERINGS ON THE TEMPERATURE OF CONCRETE EXPOSED TO THE SUN'S RAYS; TESTS MADE APRIL 3, 1933.

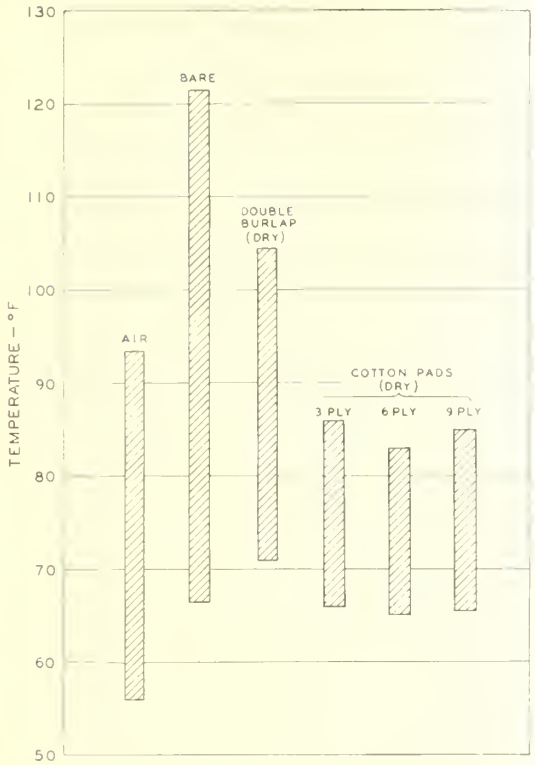

FIGURE 7.—TWENTY-FOUR-HOUR RANGE IN TEMPERATURE OF CONCRETE COVERED WITH VARIOUS MATERIALS AND EXPOSED TO THE SUN'S RAYS; TESTS MADE APRIL 3, 1933.

be found. Actually the mean temperature of the covered slabs was approximately 80 percent of that of the bare slabs, irrespective of the position of the wetted surface of the covering mats.

SINGLE-PLY COTTON MAT PROTECTS FROM SUN'S RAYS

Two new mats were made containing 1 and 2 plies of cotton, respectively, but otherwise identical to the 3-, 6-, and 9-ply mats used in the first tests. The temperature ranges for slabs covered with 1-, 2-, and 3-ply cotton mats over a 24-hour cycle compared with that of a bare slab are shown in figure 8. These data indicate that, so far as heat absorbed from the sun's rays is concerned, even a single ply of cotton offers effective protection. The difference in the insulating value of the various thicknesses of cotton mat is not shown to the best advantage under rapidly changing temperature conditions such as those to which a concrete slab is ordinarily exposed. While the additional insulation offered by the thicker mats creates a time-lag effect which is perceptible in all of the data, it would be more marked if the time during which the temperature cycle occurred were greater.

EFFECT OF COLOR OF CEMENT STUDIED

Mention was made in the report of the first series of tests of the fact that the cement used in the construction of the test slabs was of an unusually dark blue-gray or black color, and it was suggested that this dark color might affect the heat absorption by the concrete. Two duplicate slabs were constructed identical in every way except as to the color of the cement. In one a pure white and in the other a light gray portland cement were used. Because of the influence of the colors of the aggregate the resulting color was about the same in both of these slabs. However, both were considerably lighter in color than the first specimens made. The slabs containing the three cements when exposed to the sun (without a glass cover) showed the following temperature range:

Color of cement	24-hour temperature range	
	May 2	May 19
	F.	F.
White	79–98	51–92
Light gray	80–98	72–92
Dark blue-gray	78–110	72–106

Under the very moderate conditions which prevailed at the time of these observations the concrete containing the dark-colored cement reached a maximum temperature 12° to 14° F. above that of the concrete containing the light-colored cements. Since all three reached the same minimum temperature this difference of 12° to 14° in the maximum value also represents the difference in the 24-hour temperature range between slabs made with light and those made with dark-colored cement, under the conditions which obtained at the time of these tests.

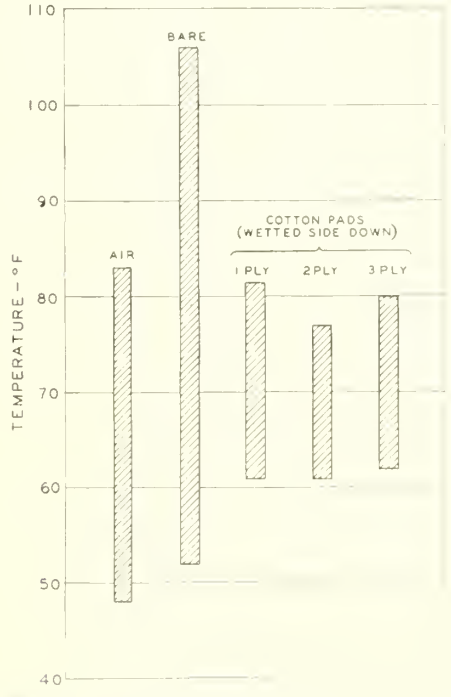

FIGURE 8.—TWENTY-FOUR-HOUR RANGE IN TEMPERATURE OF CONCRETE COVERED WITH 1-, 2-, AND 3-PLY COTTON MATS AND EXPOSED TO THE SUN'S RAYS; TESTS MADE MAY 19, 1933.

COTTON CLOTH AND WHITEWASH TRIED

Because of the opportunity afforded, observations were made of the effect on slab temperature of two other coverings which are of interest. One of these was a relatively thin white cotton cloth laid directly on the surface of the concrete in a dry state, thus representing the limiting condition in the reduction in thickness of the cotton mats. The other was a whitewash coating given the bituminous coated slab. The tests of these coverings were made in the exposure cabinet under glass.

The 24-hour temperature range for these two specimens is compared with the range for the bare slab and

one covered with a 3-ply cotton mat in figure 9. The range of the slab covered with the 3-ply cotton mat is shown to be 10° F., or 20 percent less than that of the specimen covered with the cotton cloth; 21° F., or 35 percent less than that of the bituminous coated white-washed slab; and 34° F., or 47 percent less than that of the bare concrete slab.

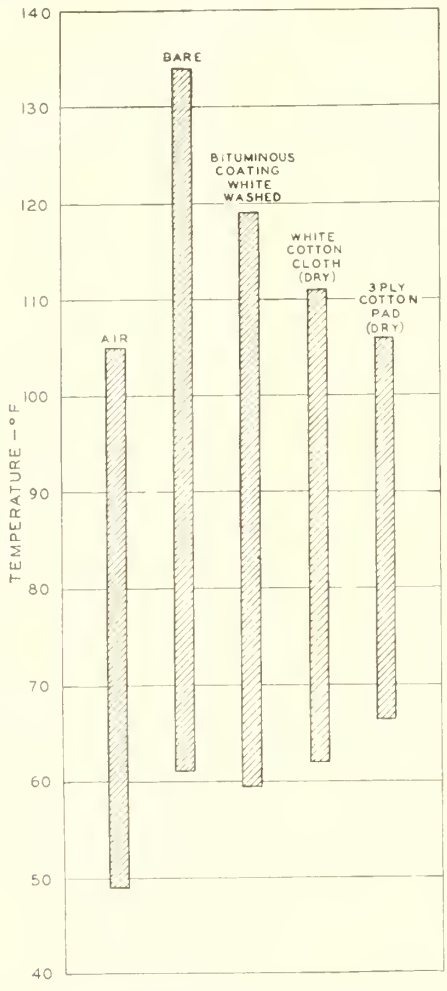

FIGURE 9.—TWENTY-FOUR-HOUR RANGE IN TEMPERATURE OF CONCRETE COVERED WITH VARIOUS MATERIALS AND EXPOSED TO THE SUN'S RAYS; TESTS MADE APRIL 24, 1933.

TESTS FOR STRENGTH AND MOISTURE LOSS

First series.—This portion of the report presents the results of a series of tests made to determine the flexural strength at 28 days of mortar specimens cured for 3 days under 3-ply, 6-ply, and 9-ply cotton mat, as compared with the strength developed by specimens cured with a double thickness of burlap kept wet during the same period. Measurements to determine the rate of moisture loss occurring under the various coverings were also taken.

The following methods of retaining moisture were studied:

Double thickness of burlap, kept wet 3 days.
Double thickness of burlap, wet once when applied.
Three-ply cotton mat, wet on one side when applied.
Six-ply cotton mat, wet on one side when applied.
Nine-ply cotton mat, wet on one side when applied.
Three-ply cotton mat, dry when applied.
Six-ply cotton mat, dry when applied.
Nine-ply cotton mat, dry when applied.

TEST PROCEDURE DESCRIBED

Test specimens consisted of mortar mixed in the proportion of 1 part cement to 2 parts sand by weight, with 14 percent water. This produced a mortar of such consistency that a film of free water showed on the surface of the specimens shortly after molding. The specimens were approximately 11 inches in length by 6½ inches in width by 2 inches in thickness and were molded in water-tight pans of sufficient depth to take the specimen and the curing mat. Mats were applied after the specimens had been exposed to the air of the laboratory for approximately 2½ hours after molding.

All of the curing mats except the double thickness of burlap, kept wet 3 days, were cut to fit the pans and the edges sealed with paraffin to prevent moisture loss except through the top. In the case of the wet burlap the covering material was cut longer than the specimen and the free end was immersed in water. This provided an ideal curing condition for the 3-day period. The results of the tests made with this mat are used as the standard of comparison.

The mats which were applied wet were thoroughly sprinkled with water on one side immediately prior to application. They were applied with the wet side down. The same comments as regards the penetration of mositure into the mats which were given in connection with the discussion of the temperature tests apply in this case also.

At the end of 72 hours (3 days) the curing mats were removed. The specimens were weighed and then allowed to remain in the pans with the surface exposed to the air of the laboratory until the age of 26 days. They were then removed from the pans, immersed in water for 2 days and tested for flexural strength on a 9-inch span with the load applied at the center of the span. The slabs were tested in the flat position, with the upper surface (as cast) in tension. Changes in moisture were obtained by weighing the specimens at the end of 3, 10, 20, and 26 days, as well as immediately before testing at 28 days.

Three rounds of tests were run, eight specimens to each round, making a total of 24 specimens.

The results of these tests are given in tables 2 and 3 and in figures 10, 11, and 12.

Table 2 gives the percentage of moisture retained by each specimen at 3, 10, 20, 26, and 28 days, as compared with the original water content. Average results for the three rounds are shown graphically in figure 10. Figure 11 gives the 3-day test data in bar diagram form to facilitate comparisons with the strength data.

The results of the individual strength tests are shown in table 3, with the average results expressed also as percentages of the strength developed by the use of the double thickness of burlap kept wet 3 days.

RESULTS OF TESTS DISCUSSED

In figure 10 the moisture content of each group of specimens at 3, 10, 20, 26, and 28 days is shown. In plotting the values in this figure as well as in figures 11 and 12, the three rounds of tests have been averaged. In figure 10 the results have been further averaged so as to give for each condition of curing the average value for the three thicknesses of cotton. This was done because the individual points were so close that they could not be very well plotted separately.

From figure 10 the effectiveness of each of the curing mats is immediately apparent. For instance, at the

TABLE 2.—*Percentage of original water content of specimens at ages indicated (3-, 6-, and 9-ply mats)* [1]

Curing method	Specimen no.	Percentage of original water content at age of -				
		3 days	10 days	20 days	26 days	28 days
Burlap, kept wet 3 days	1	101.4	90.4	87.6	86.8	97.5
	2	98.1	88.4	85.3	84.9	93.8
	3	100.4	90.2	87.3	86.1	95.6
Average		100.0	89.7	86.7	85.9	95.6
Burlap, wet when applied	1	86.5	82.1	79.6	78.9	99.9
	2	84.7	82.0	79.4	78.7	92.5
	3	89.2	85.1	82.5	81.5	100.6
Average		86.8	83.1	80.5	79.7	97.7
3-ply mat, wet when applied	1	102.5	91.7	88.5	87.6	98.5
	2	99.6	90.0	87.1	86.4	95.4
	3	100.5	90.7	87.8	86.6	95.7
Average		100.9	90.8	87.8	86.9	96.5
6-ply mat, wet when applied	1	100.3	90.3	87.0	86.2	96.4
	2	100.7	91.0	87.8	87.2	96.8
	3	102.4	92.0	88.6	87.4	97.4
Average		101.1	91.1	87.8	86.9	96.9
9-ply mat, wet when applied	1	100.6	91.1	88.3	87.4	97.2
	2	99.0	90.0	87.0	86.4	95.1
	3	101.2	91.4	88.3	87.0	96.7
Average		100.3	90.8	87.9	86.9	96.3
3-ply mat, dry when applied	1	92.7	86.4	83.5	82.9	101.9
	2	91.1	85.8	83.1	82.2	93.3
	3	92.0	86.5	83.5	82.3	97.6
Average		91.9	86.2	83.4	82.5	97.6
6-ply mat, dry when applied	1	93.1	86.4	83.9	83.1	98.5
	2	91.3	86.0	83.0	82.7	95.2
	3	92.7	87.5	84.7	83.3	96.9
Average		92.4	86.6	83.9	83.0	96.9
9-ply mat, dry when applied	1	93.5	86.8	83.9	82.8	96.2
	2	92.2	86.8	83.9	83.0	100.0
	3	93.0	86.9	83.9	83.0	94.8
Average		92.9	86.8	83.9	82.9	97.0

[1] Curing agents removed at end of 3 days, and specimens cured from 3 to 26 days in laboratory air. Specimens immersed in water at end of 26 days.

TABLE 3.—*Results of tests for flexural strength at the age of 28 days (3-, 6-, and 9-ply mats)*

Curing method	Modulus of rupture, in pounds per square inch				Strength ratio [1]
	(1)	(2)	(3)	Average	
Burlap, kept wet 3 days	904	893	948	915	1.00
Burlap, wet when applied		814	801	808	.88
3-ply mat, wet when applied	917	988	904	936	1.02
6-ply mat, wet when applied	910	919	919	916	1.00
9-ply mat, wet when applied	926	934	902	921	1.01
3-ply mat, dry when applied	808	919	756	828	.90
6-ply mat, dry when applied	783	776	820	793	.87
9-ply mat, dry when applied	841	799	812	817	.89

[1] Percentage of strength developed by double burlap kept wet 3 days.

end of the 3-day curing period the specimens cured under burlap which was kept wet and those cured under cotton which was wet once before being applied showed moisture contents of at least 100 percent of the original mixing water. This applies to all three thicknesses of cotton mat. In view of the fact that some of the original mixing water was lost during the 2½ hours before the mats were applied, this means that the specimens actually took up water from the mats during the 3-day curing period. The burlap which was wet when applied but which was not subsequently wet was the least effective in retaining moisture, the specimens at 3 days retaining but 87 percent of the original mixing water. The mats which were applied dry gave intermediate results, the average for the three thicknesses

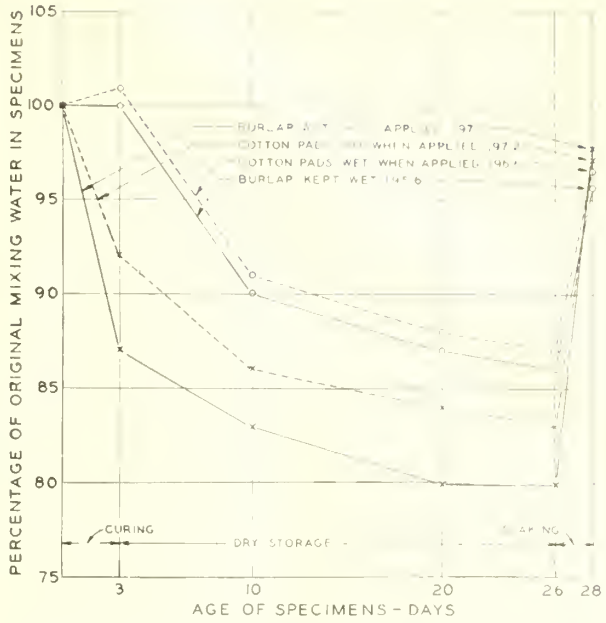

FIGURE 10.—COMPARATIVE MOISTURE CONTENTS OF MORTAR SPECIMENS CURED UNDER BURLAP AND COTTON.

being 92 percent. The relative order of moisture loss is the same for the entire dry-storage period, the specimens continuing to lose moisture up to 26 days. However, at 28 days, after 2 days immersion in water, it was found that the relative order of moisture loss had been reversed, both the specimens cured under burlap wet when applied and those cured under the dry mats showing slightly higher moisture contents than the specimens cured under burlap kept wet and the mats which were wet when applied. This is an indication of the greater curing efficiency of the wet cotton mats which resulted in somewhat denser specimens, lower moisture loss at 26 days, and less gain in weight between 26 and 28 days.

Figure 11 shows the relative moisture losses at 3 days in bar diagram form. Here the values for the three thicknesses of mat are shown separately. The arrangement of this chart affords a good opportunity to study the relative order of values as compared with the burlap which was kept wet for 3 days and which is considered as the standard for comparison.

Figure 12 shows the flexural strength at 28 days for each curing condition plotted as a percentage of the flexural strength of the standard cured specimens (burlap kept wet 3 days). The order of values agrees very well with the percentage of moisture retained in the specimens at the end of the 3-day curing period, indicating a rather definite relation between strength and the amount of curing received during the first 3 days.

These data indicate that, from the standpoint of strength, the 3-ply, 6-ply, and 9-ply cotton mats, if wet when applied, are as effective from the standpoint of curing as a double thickness of burlap kept wet for 3 days. Furthermore, all three thicknesses give comparable results insofar as strength is concerned. The results also indicate that curing with mats applied dry is not as effective as when the mats are wet before being applied. The dry mats developed only 88 percent of the strength attained when they were applied wet, which was about the same relative strength as that obtained with burlap wet once when applied.

Second series.—In view of the fact that the 3-ply mats gave as good results from the standpoint of moisture

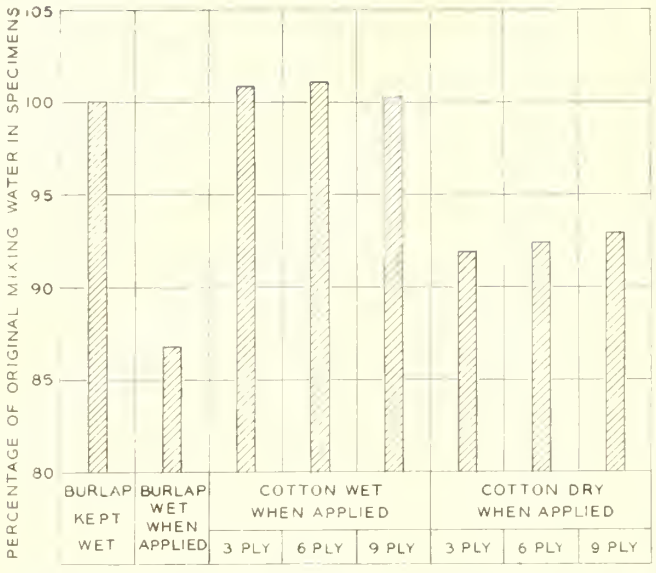

FIGURE 11.—COMPARATIVE MOISTURE CONTENTS OF MORTAR SPECIMENS AFTER 3 DAYS' CURING UNDER BURLAP AND COTTON; FIRST SERIES

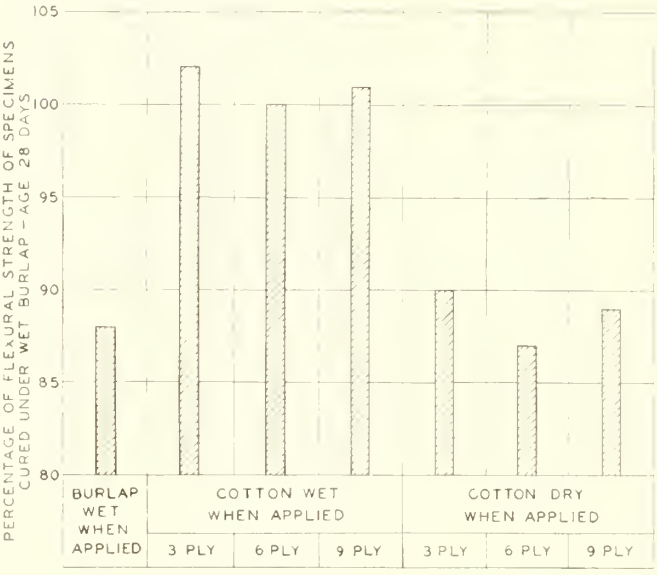

FIGURE 12.—COMPARATIVE STRENGTHS OF MORTAR SPECIMENS CURED UNDER BURLAP AND COTTON; FIRST SERIES

retention and strength as the 6- and 9-ply mats, it was decided to repeat the tests, using mats made up of 1 and 2 plies of cotton. In order to tie in the results with the previous work, tests of the 3-ply mat as well as the double burlap, kept wet 3 days, were repeated. The same test procedures were followed throughout the supplementary series, the only variation being the thickness of the mats.

The results of these further tests are shown in tables 4 and 5 and in figures 13 and 14.

Table 4 gives the percentages of moisture retained by the cotton mats at the various test periods. Comparison with table 2 will show that the values are almost identical with the corresponding values for the 3-, 6-, and 9-ply mats shown in table 2. This indicates quite definitely that the 1- and 2-ply mats are just as effective in retaining water as the 3-ply and thicker mats, within the limits of these tests.

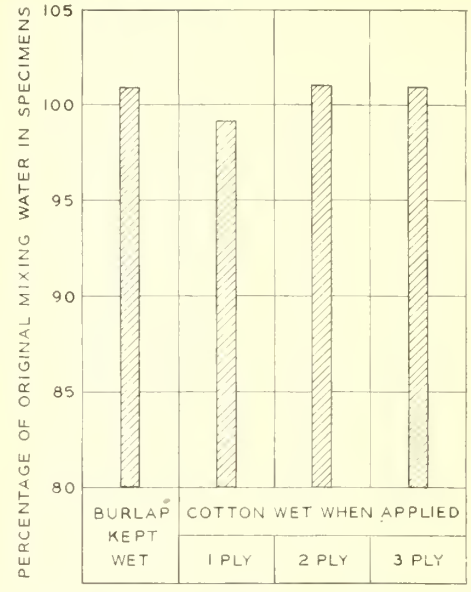

FIGURE 13.—COMPARATIVE MOISTURE CONTENTS OF MORTAR SPECIMENS AFTER 3 DAYS' CURING UNDER BURLAP AND COTTON; SECOND SERIES.

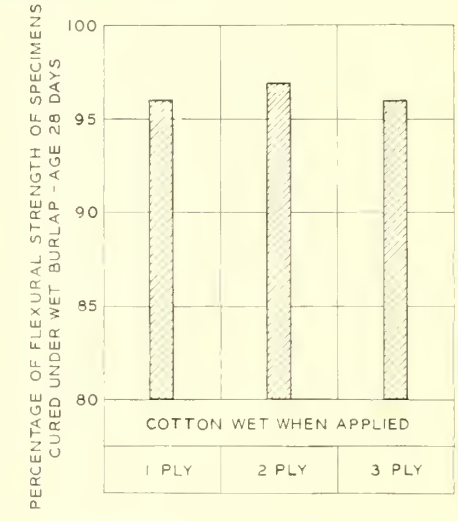

FIGURE 14.—COMPARATIVE STRENGTHS OF MORTAR SPECIMENS CURED UNDER BURLAP AND COTTON; SECOND SERIES

In table 5 the flexural strengths of mortar specimens cured under 1-, 2-, and 3-ply mats, as well as under burlap kept wet 3 days, are shown, together with the strength ratios of the cotton-cured specimens in terms of the strength developed by the burlap cure. The three thicknesses, it will be noted, all show substantially the same efficiency—that is, either 96 or 97 percent of wet burlap. Reference to table 3 will show that, in the initial series of tests, the 3-ply mats as well as the 6- and the 9-ply showed about 100 percent of the burlap cure. Dry mats, on the other hand, only gave about 90 percent of the standard. The slightly lower ratio shown by the 3-ply mats in the second series (table 5) is probably due to the somewhat high strength developed by the burlap-cured standard of comparison as compared to the strength obtained in the first series (table 3). The fact that, in each group, the strengths of the wet mats of different thicknesses are substantially the same leads to the conclusion that for all practical purposes the 1- and 2-ply mats are just as effective as

(Continued on p. 92)

AN INDEX OF THE COST OF HIGHWAY CONSTRUCTION

Reported by J. L. Harrison, Senior Highway Engineer, Division of Management, United States Bureau of Public Roads

IN ORDER to place a comprehensive statement of the relative purchasing power from year to year of the funds made available for highway construction at the disposal of those responsible for the determination of policy in this field, and of others who may be interested, the division of management of the Bureau of Public Roads has prepared indexes of such construction. These indexes are shown in figures 1 and 2.

The indexes given in figure 1 show the trend in unit prices for excavation, surfacing, and structures over the period from 1922 to 1933 and a similar price trend for a "composite mile" of road. Figure 2 gives the trend of the usage index and the cost index for a mile of road for the years from 1923 to 1931, as well as the price trend for the composite mile.

It is well known that the quantities of materials and the amount of work required to build a mile of road have gradually increased during the last dozen years. The usage index shows this trend. Since the price index is based on the same quantities of excavation, surfacing, and structures for each year, it does not give a true picture of the variation in the cost of road construction. The cost index takes usage into account, and therefore shows the trend in actual cost per mile.

In the determination of these indexes it was necessary to select the unit in which values are to be expressed, the classes of work to be included, the weight to be given to each, and a reference base period. It was decided that the index figure should be developed on the basis of a composite mile of highway and that the classes of work to be included and the weights for each should be as follows:

Common excavation	cubic yards	17,491
Pavement	square yards	3,726
Reinforcing steel	pounds	16,000
Structural steel	do	1,325
Structural concrete	cubic yards	68

The advantage of a base made up in this way lies in the fact that it measures trends by means of items that are so widely and so frequently used that regional trends in price as well as national trends can be determined with a reasonable degree of accuracy, even on a quarterly basis.

It was decided to use the fiscal years 1925 to 1929, inclusive, as the base period. This is an arbitrary selection, though it produces weightings that, as an expression of usage of materials, are about intermediate between the weightings that would result if the usage of 1922 or the usage of 1931 had governed. A 5-year base period was used in order more accurately to reflect the influence of the various States on both prices and quantities.

The index figures here shown were developed on the basis of the average cost of 1 mile of highway composed of units of grading, surfacing, and structures in the same ratios of use as were found in the highway construction program for the years comprising the base period.

INDEXES SHOW TREND IN COST OF HIGHWAY CONSTRUCTION

The results of this study show that unit prices for highway excavation, surfacing, and structures declined steadily from 1923 to the second quarter of 1932. Since that date a definite upturn in unit prices has been noted. The combined effect of these trends is shown in the index of the cost of a composite mile of highway, which is based on the assumption of equal quantities of excavation, surfacing, and structures per mile in each year. The price index curves are shown in figure 1.

The quantities of work and materials required per mile of highway have increased almost continuously during the years from 1923 to 1931. The variation of the usage index, which gives the trend in the cost of a composite mile of highway, on the assumption of constant unit prices, is shown in figure 2.

The cost index, which takes into account both the decline in unit prices paid for excavation, surfacing, and structures and the increasing quantities of these items used per mile, is also shown in figure 2. Between 1926 and 1930 the cost trend was upward. In 1931 the decline in price levels was felt sufficiently to cause a drop in the cost index as compared with that of the previous year.

CHANGES IN CONSTRUCTION PRACTICE MAKE DETERMINATION OF INDEXES DIFFICULT

In the presentation of the relative purchasing power from year to year of the money expended in any field, a fixed base from which to measure this purchasing power is the first essential. Actually this requirement cannot be met fully, for in every field of human endeavor though the physical standards of measurement as applied to the commodities used in this field remain constant the methods and practices by which those selected for use as a base are produced change constantly, often radically. This is particularly true in the highway field where the development which has taken place during the past 10 years has been both extensive and far-reaching.

This is a condition by no means confined to the highway field. It is, for instance, perfectly apparent that an average pair of shoes is not the same thing today that it was 20 years ago. Though it is equally true, it is not so apparent that even a pound of the leather out of which these shoes are made is a different thing now from what it was then. The processes by which it is produced have changed, and many of the chemicals used in its treatment have been modified, at least as to the methods by which they are prepared.

Any number of additional illustrations of this situation could be given but to no very valuable purpose, for such illustrations would merely add emphasis to the statement already made that, even though the apparent physical aspects of the commodities which are selected

FIGURE 1.—PRICE TREND IN HIGHWAY CONSTRUCTION FROM 1922 to 1933. AVERAGES FOR 1925 TO 1929 TAKEN AS BASE

as a base from which to measure changes in the purchasing power of money remain the same, these commodities are not thereby insured a constant internal composition or constancy as to the methods, processes, and practices by which they are created. Insofar as either their internal composition or the methods by which they are created are altered, they necessarily fall short of forming an absolutely fixed base. However, in an age that has been marked by rapid changes in the materials in common use, to say nothing of changes in the methods and practices used in combining them or the machines used in making them, the multiplicity of these changes precludes the possibility of constructing any base which altogether avoids them and so remains, in fact, fully constant over extended periods of time.

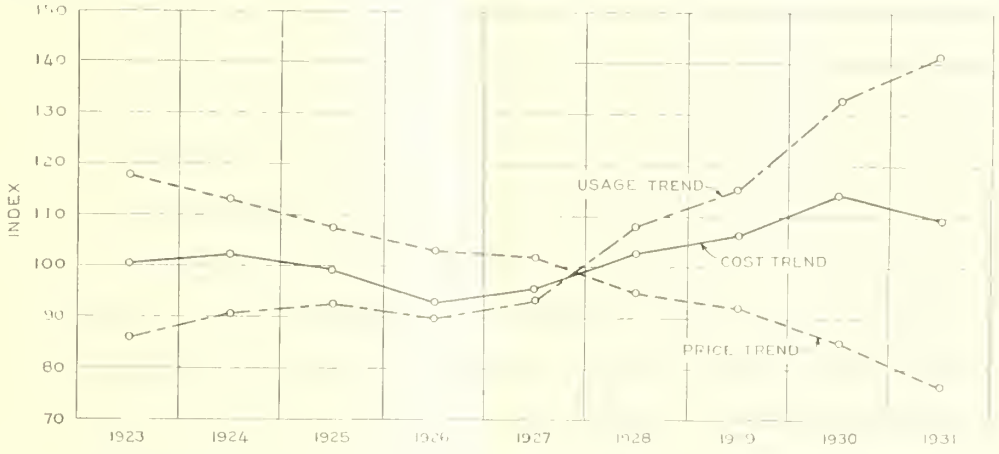

FIGURE 2.—VARIATION IN INDEXES OF HIGHWAY CONSTRUCTION COST FROM 1923 TO 1931. PRICE INDEX SHOWS TREND IN COST OF COMPOSITE MILE COMPOSED OF SAME QUANTITIES OF EXCAVATION, SURFACING, AND STRUCTURES IN EACH YEAR. USAGE INDEX SHOWS TREND IN QUANTITIES OF EXCAVATION, SURFACING, AND STRUCTURES USED PER MILE IN EACH YEAR. COST INDEX, TAKING ACCOUNT OF USAGE, SHOWS TREND IN ACTUAL COST PER MILE. AVERAGES FOR 1925 TO 1929 TAKEN AS BASE.

In the highway field this difficulty must be faced in a somewhat more aggravated form than in other fields, since, in addition to the numerous internal changes which have taken place in the great bulk of the commodities used in constructing highways there have been numerous changes in the physical form of the units by which highway work is measured, more particularly in the paving field. To illustrate, in concrete paving construction the standard unit of measurement is the square yard of finished pavement, but the only fixed thing about this unit is its finished area. The specifications that govern the selection of all of the materials entering into the composition of the completed unit are subject to frequent changes, usually minor, it is true, but nevertheless changes. The proportions in which these materials are used, the amount of manipulation that is required, the exactness with which the surface must be finished, finally even the thickness of the pavement — all change from time to time.

Admittedly, this is rather an extreme example of the instability of what, in theory, should be stable units, and it cannot be denied that this sort of instability subtracts something from the authority of any conclusions index figures would otherwise appear to justify. But no better means of measuring relative prices or their reciprocal, the relative purchasing power of money, is available than is offered by the commodities with which an industry deals, and no better measure of them is to be had than the units on which business transactions are based. These units have therefore been used in developing an index to show the relative purchasing power of the money spent in the highway field, and it is believed that a substantially accurate indication of the relative purchasing power of the money used in it results.

In further comment in regard to this matter it may be remarked that if, in fact, the relative purchasing power of the money spent on highway construction differs from the purchasing power as indicated by the index, there is every reason for believing it to be slightly greater than as shown. This statement is based on the fact that the majority of the changes which have taken place in the composition of the units that have been used in establishing the index is in the direction of improved quality or increased quantity, or of both. As these factors tend to produce extra cost — seldom to subtract from the cost — it should be, and probably is, true that when the purchasing power of money is rising their effect is to show somewhat less of a rise than actually takes place, and that when the purchasing power of money is falling their effect is to accentuate the fall. In short, the effect of these factors has been a little to understate rather than to overstate the relative purchasing power of the money used in the highway field.

WEIGHTING THE INFLUENCE OF UNITS

Under some circumstances an index figure that is based on unweighted prices appears to yield results which prove entirely satisfactory; but in the highway field an index developed in this way would be open to the objection that so much more of some kinds of material, such as concrete, is used than of others, such as steel, that unless the amount used is considered, the index is likely to give an unreliable indication of relative purchasing power, for changes in price seldom affect all commodities in the same degree. It is not uncommon for some commodities to rise in price at the same time that others are falling. Then, too, freight charges, which are a large factor in the cost of some types of construction, are an almost negligible factor in the cost of others.

To illustrate these points: If, during the base period and during a succeeding year, prices on four items have been as follows, the use of the unit price alone indicates a rise in the index number — that is, a drop in the purchasing power of the money expended.

	Base period	Another year
Concrete pavement	$2.00	$1.90
Brick pavement	3.50	4.00
Macadam surfacing	1.00	1.15
Gravel surfacing	.50	.40
Total	7.00	7.45
Index number	100.0	106.4

If, instead of unweighted prices, a weighting based on use is introduced, the result may be quite different, as the following illustration (weightings assumed) will indicate.

	Weighting	Base year		Another year	
		Unit price	Total	Unit price	Total
Concrete pavement	10	$2.00	$20.00	$1.90	$19.00
Brick pavement	1	3.50	3.50	4.00	4.00
Macadam surfacing	2	1.00	2.00	1.15	2.30
Gravel surfacing	12	.50	6.00	.40	4.80
Total			31.50		30.10
Index number			100.0		95.5

Where the number of units on which an index is based is large, and more particularly when the index is based on a wide variety of commodities, the unweighted index is perhaps as reliable as the weighted index; but in a restricted field from which it is necessary to select a limited number of items, weighted figures appear likely to yield more reliable results if reasonably accurate weightings can be given to the units selected.

In theory the weighted figures and the unweighted figures are about equally responsive to general price movements. Periods of inflation and deflation tend to affect all commodities about alike, and their effect should be made about equally apparent by both styles of index figures. The same should be true of any general alteration in wage levels. But when it comes to the matter of reflecting the effect on cost of such factors as overproduction, underproduction, more complete mechanization, and improved efficiency, index figures based on weighted prices have a distinct advantage.

Changes in mechanical devices or in processes and improvements in efficiency—to say nothing of overproduction or underproduction—cause radical changes in price. In a restricted field the full effect of changes of this sort may easily be overlooked unless weighted figures are used in developing the index; for each change of this sort is likely to be of limited application, with the result that while in a broad field it has only a limited effect, in a narrow field it may have either a powerful effect or almost no effect at all—the result depending on whether commodities of major or of minor importance in that field are affected. For these reasons it has been thought wise to use weightings in preparing indexes for the highway construction field.

COMPOSITE MILE TAKEN AS BASIS FOR WEIGHTING

If weighted figures are to be used, a basis for the weightings must be determined. After some study it was decided that a composite mile of highway—that is, a mile in which the total yardage of surfacing is appropriately distributed among the various types commonly used, and to which average amounts of grading, structural concrete, and steel have been assigned—would be used as the underlying basis for all weightings. The Bureau's records (compiled from reports of State authorities) of State highway mileage built during the years from 1923 to 1931, inclusive, were used to determine the component types of surfacing entering into the composite mile of pavement.

On the other hand, while the Bureau has satisfactory statistics covering such matters as the mileage of work completed on the State highway systems and the types laid down, no records of such matters as the yardage of excavation handled, the yardage of concrete used in structures, etc., are available except for the work in which the Federal Government has participated. The lack of information of this sort effectively prevented the development of weightings wholly based on the general State highway practice. Careful consideration of these matters led to the conclusion that a satisfactory result might be attained if the composition of the composite mile of pavement were based on average usage in construction work on the State highway systems, and the amounts of the correlated items (grading, structural concrete, and steel) were based on the Bureau's experience on Federal-aid construction.

The propriety of determining the composition of the base in this way necessarily depends on an assumption that practices in the State highway field, taken as a whole, are substantially the same as prevail in the somewhat more restricted field in which Federal aid is applied. That they must be is apparent from the fact that governing specifications are the same in both fields and that design is by the same organizations.

To further emphasize the correctness of this assumption, one has but to remark the fact that as he travels today he finds nothing that will afford any indication as to whether Federal aid was used on the highway at the particular point where he happens to be or whether it is a part of the broader field the State has covered without Federal assistance.

As a result of this decision, the following items were selected for inclusion in the base for these indexes. Their further adjustment and the weightings assigned to them are considered in subsequent paragraphs.

From the Bureau's Federal-aid records:
- Excavation:
  - Common
  - Unclassified
  - Rock
- Structures:
  - Reinforcing steel
  - Structural steel
  - Structural concrete, class A
  - Structural concrete, class B
  - Structural concrete, class C

From statistics on State highway mileage:
- Pavement:
  - Gravel and sand-clay
  - Macadam
  - Bituminous macadam
  - Bituminous concrete
  - Portland cement concrete
  - Brick

These items cover somewhat more than 90 percent of the total cost of highway construction. Therefore, though their number is not great, it would appear to be adequate. The items not used involve the use of about the same basic commodities, manufacturing processes, transportation problems, and the same classes of labor which were involved in the items used. To include them would complicate the calculations but would neither clarify nor improve the result.

MINOR ITEMS CONVERTED INTO EQUIVALENT AMOUNTS OF MORE IMPORTANT ITEMS

The various items that enter into a composite mile of pavement and the quantity of each to be used having been determined, a question naturally arises as to the distribution of these items—that is, are they universally and frequently used, or are some or perhaps most of them used locally or regionally and infrequently? If the latter is the case, can substantial accuracy be obtained if the items which are used locally or regionally, and on the whole infrequently, are converted into an equivalent amount of items more widely used but likely to show similar price trends?

In answer to the first question, it has been found that of the three grading items common excavation is widely used; unclassified excavation is considerably used but in many States not used at all; and rock excavation, though widely used, is not so frequently used as might be supposed. All of these items involve about the same construction practices and show a similar price trend. In general, the price of unclassified excavation is about one and a half times the price of common excavation, while the cost of rock excavation tends to run about three times the cost of common.

Under surfacing items, examination indicates that gravel, sand-clay, and topsoil are local in their general aspects, generate little freight, and in construction methods as well as in the nature of equipment used are similar to grading operations. On this account the natural conversion of gravel, sand-clay, and topsoil surfacing is into units of common excavation. As macadam and bituminous macadam occupy a middle ground, macadam has been converted into grading units and bituminous macadam into concrete pavement, the latter type having been selected for the conversion of all of the higher types of pavement. These conversions are shown in table 8.

In this way the 3 grading items and the 6 paving items indicated above are reduced to 2 items, common excavation and concrete pavement, the weightings of these 2 items being altered to reflect the influence of the various items that have been dropped. For similar reasons structural concrete, classes B and C, were merged with structural concrete, class A.

The determination of the primary weightings is discussed in succeeding paragraphs, the purpose being to indicate the methods that have been used in working out these weightings.

Excavation.—After it had been determined that the primary weightings should be based on 1 composite mile of highway, that the years 1925 to 1929 should be used as the base period, and that the units should be those already noted, the first problem that required solution was the amount of excavation (grading) to be assigned to this composite mile.

Three conditions required evaluation: First, under the Bureau's policy of encouraging stage construction, a considerable mileage of road is graded during one construction season and surfaced during some subsequent construction season. During the earlier years in which Federal aid was granted for highway construction, this policy resulted in a rapid statistical accumulation of graded and drained but unsurfaced mileage. Though this policy has not changed, the rate at which graded and drained roads are surfaced is more nearly in harmony with the rate at which new grading and draining is done.

The second condition to be evaluated is the importance of the apparent accumulation of graded and drained but unsurfaced mileage. In all probability this is at least largely a statistical accumulation, arising from the fact that graded and drained mileage often is surfaced by the States without the assistance of Federal aid, and therefore without subtraction from the Bureau's recorded accumulation of unsurfaced mileage. In the same way Federal aid is often used in surfacing roads which have been graded and drained without the assistance of Federal aid.

There is, of course, a large mileage on which both the grading and draining and the surfacing are done at the same time. On account of this situation, which for various reasons does not lend itself to a more exact solution, it was decided that the whole amount of the grading done during the base period would be allocated to the mileage surfaced during that period. This is, in effect, an assumption—reasonably accurate, we believe—that in the present advanced stage of our construction program partially completed highways are being advanced toward completion at about the same rate that partially completed mileage is being created.

The third condition that has to be evaluated arises from the fact that the Bureau's accumulation of bid-price data is not quite as complete as its data on mileage of work completed. There are several reasons for this. The first is that it would be a very complex statistical problem to so adjust the number of units entering into the Bureau's bid-price data on any class of work that at the end of the year these units would total to an exact agreement with the mileage of pavement built. As such an agreement would add nothing to the value of the bid-price data, as now obtained, no effort has been made to bring about such an agreement.

A second and perhaps even more important reason for the fact that the quantities on which bid prices have been obtained differ from those indicated by the mileage constructed is found in the fact that the preliminary estimates of quantities on which bids are taken commonly differ somewhat from the quantities actually handled. Finally, it is probable that the Bureau's bid-price data does not include quite all of the projects on which work has been done.

Structures.—The development of a highway system always requires the erection of a good many structures, principally bridges and culverts. In the development of this system of index figures the structural field is covered by the inclusion of the following items: Reinforcing steel, structural steel, and structural concrete. There are many other items, such as pipe (concrete, cast iron, corrugated iron, terra cotta), excavation of various grades, special work on handrails, etc., that this list does not include; but it does include the items of greatest importance. Most of the excluded items are, in nature, quite similar to those that are included.

Reinforcing steel.—Reinforcing steel is a component part of reinforced concrete pavement and of structural concrete, class A. It may or may not be used in structural concrete, classes B and C, depending entirely on design policy and its application to specific cases. On this account, as a matter of theory, reinforcing steel should not appear as a separate item in the list on which the index is based. In practice, however, though some States include reinforcing steel in such items as class A concrete, more call for separate bids on steel, these bids including the cost of handling the steel and placing it in the finished work.

Bids taken in this way may not indicate whether the steel is to be used in the pavement or in the structure. On this account, although shown as an item in the

structural list, a part of the total is in the nature of a correction of deficiencies in the amount included in the assigned yardage of reinforced concrete pavement. In view of the fact that some of the steel known to be used is not included in the recorded quantities, the reinforcing steel component in the base was taken as 16,000 pounds, an increase of 69 percent over the average quantities reported for the base years 1925 to 1929. The resulting adjustments are shown in table 12.

CORRECTION FACTORS APPLIED TO QUANTITIES USED IN DETERMINING UNIT BID PRICES

The fact that the mileage of pavement constructed and the number of units of grading and other items on which average bid prices have been recorded are not in agreement has made it necessary to use a correction factor. To obtain this factor it has been assumed that the processes by which the quantities used in developing unit bid prices are accumulated result in a uniform rate of underestimate applicable to all of the items listed (except surfacing items based on tables of State highway mileage).

As concrete pavement appears to offer the best basis for determining the correction factors, it has been used. The Bureau's records of the mileage of concrete roads on which Federal-aid funds have been used are complete and accurate. To convert the known mileage into an equivalent yardage involves a knowledge of average widths. The division of design has made several excellent studies touching on this matter. From the data given in these studies and from other known facts, it appears that during the period covered by these indexes the average width of the pavements built has increased pretty uniformly at the rate of about 0.3 foot per year to the currently prevailing average of 20 feet. On this basis, the recorded mileage of concrete pavement has been converted into an equivalent yardage, and with this yardage as a base the correction factors to be applied to the grading and other items used in the composite mile (except surfacing) have been computed. The details of the development of the correction factors are given in table 1.

TABLE 1.—*Factors for adjusting reported quantities*

Year	Miles of concrete pavement built [1]	Width of slab, in feet	Square yards per mile	Square yards of concrete pavement [2]	Square yards of concrete pavement reported [3]	Correction factor
1923	1,440.3	17.6	10,325	14,871,097	16,492,245	90.17
1924	1,667.9	17.9	10,501	17,514,618	18,750,167	93.41
1925	2,806.4	18.2	10,677	29,963,933	20,713,140	144.66
1926	2,660.0	[4] 18.5	10,853	28,868,980	24,600,841	117.35
1927	2,284.1	18.8	11,029	25,191,339	23,548,060	106.98
1928	3,114.6	[4] 19.1	11,205	34,899,093	34,415,209	101.41
1929	3,131.3	19.4	11,381	35,637,325	26,402,583	134.98
1930	3,170.3	19.7	11,557	36,639,157	26,118,498	140.28
1931	4,540.7	[4] 20.0	11,733	53,276,033	46,845,087	113.73

[1] From Federal-aid records.
[2] Total miles multiplied by square yards per mile.
[3] Bid-price records.
[4] Widths for these 3 years obtained from tables compiled by the division of design. From these figures the width was assumed to have changed 0.3 foot per year.

The adjusted quantities (except surfacing) and quantities per surfaced mile are shown in tables 2 an

TABLE 3.—*Quantities of construction items per surfaced m*

Year	Mileage surfaced [1]	Excavation	Reinforcing steel	Structural steel	Stru al c.
		Cubic yards	*Pounds*	*Pounds*	*C yc*
1923	6,949.3	8,068	5,491	2,297	
1924	7,004.7	8,364	7,329	2,258	
1925	9,217.8	9,238	8,044	4,718	
1926	8,427.6	11,068	8,334	3,629	
1927	7,072.9	10,960	7,566	3,301	
1928	7,917.6	12,545	10,114	4,953	
1929	7,049.2	17,028	13,329	5,024	
1930	6,676.4	18,946	15,905	7,750	
1931	9,061.0	22,361	18,214	12,216	

[1] From Federal-aid records.

Further adjustment in the quantities of reinforc steel was made in the following manner. In table 3 find the average quantities per mile (after applicat of the correction factor) to be as follows:

	Pounds
1925	8,044
1926	8,334
1927	7,566
1928	10,114
1929	13,329
Total	47,387
Average	9,477

As stated previously, the quantity of reinforcing s was increased to 16,000 pounds to account for defici cies in the reported amounts. The adjustment fac used was $\frac{16,000}{9,477}=1.6883$. The adjusted quantities shown in table 4.

TABLE 4.—*Adjustment of quantities of reinforcing steel*

Year	Quantity of reinforcing steel	Adjustment factor	Adj qua
	Pounds		*Pou*
1923	5,491	1.6883	
1924	7,329	1.6883	
1925	8,044	1.6883	
1926	8,334	1.6883	
1927	7,566	1.6883	
1928	10,114	1.6883	
1929	13,329	1.6883	
1930	15,905	1.6883	
1931	18,214	1.6883	

QUANTITIES OF PAVEMENT TYPES IN COMPOSITE MILE BASEI REPORTS OF STATE HIGHWAY MILEAGE BUILT

The composition of the composite mile of pavem is based on mileage tables prepared by the divisio highway transport, from reports of State authorit

TABLE 2.—*Adjusted quantities of construction items, based on quantities shown in bid-price tables*

Year	Correction factor	Combined excavation		Reinforcing steel		Structural steel		Structural conc	
		Reported	Adjusted	Reported	Adjusted	Reported	Adjusted	Reported	Adju
		Cubic yards	*Cubic yards*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Cubic yards*	*Cubic*
1923	90.17	62,178,407	56,066,270	42,318,786	38,158,849	17,702,791	15,962,607	408,798	36
1924	93.41	62,720,726	58,587,430	54,960,417	51,338,526	16,934,807	15,818,803	498,819	46
1925	144.66	58,862,196	85,150,053	51,253,859	74,143,832	30,066,422	43,494,086	405,029	58
1926	117.35	79,483,545	93,273,940	59,852,629	70,237,060	26,058,778	30,579,976	488,814	57
1927	106.98	72,462,022	77,519,871	50,020,271	53,511,686	21,823,280	23,346,545	420,582	44
1928	101.41	97,941,791	99,322,770	78,963,647	80,077,034	38,668,944	39,214,176	509,860	51
1929	134.98	88,925,066	120,031,054	69,609,347	93,958,697	26,238,148	35,416,252	421,084	56
1930	140.28	90,169,860	126,490,280	75,695,757	106,186,008	36,884,909	51,742,150	581,439	81
1931	113.73	178,155,572	202,616,332	145,113,073	165,037,098	97,326,093	110,688,966	1,126,279	1,28

The reported mileage of State highways completed is shown for the years 1923 to 1931 in table 5.

In this and in subsequent tables the classification of road types is simplified. Under gravel are included sand-clay and topsoil roads, as well as treated and untreated surfaces; water-bound macadam includes both treated and untreated surfaces; bituminous concrete includes sheet asphalt; and under brick is included all block pavement.

TABLE 5.—*Reported mileage of State highways completed in the years from 1923 to 1931, compiled from reports of State authorities*

Year	Graded and drained	Gravel[1]	Water-bound macadam[2]	Bituminous macadam	Bituminous concrete[3]	Portland cement concrete	Brick and other block pavement[4]	Total	Total mileage surfaced
	Miles	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*
1923	5,814	7,852	1,017	990	511	3,878	235	20,297	14,483
1924	5,957	9,045	1,467	997	684	4,851	164	23,164	17,207
1925	5,316	10,005	1,144	1,000	565	4,975	146	23,152	17,836
1926	7,060	12,272	1,088	1,132	460	4,403	137	26,552	19,492
1927	7,151	11,132	1,707	1,105	501	5,058	69	26,723	19,572
1928	8,675	10,823	1,006	1,979	598	6,055	116	29,252	20,577
1929	7,451	14,582	1,642	1,200	556	6,991	100	32,522	25,071
1930	7,813	15,306	1,371	1,276	697	8,651	163	35,277	27,464
1931	10,095	20,573	1,453	1,939	749	9,664	161	44,634	34,539

[1] Includes sand-clay and topsoil.
[2] Includes treated and untreated macadam.
[3] Includes sheet asphalt.
[4] Includes all block pavements.

Table 6 gives the percentage distribution of the surfaced mileage by types.

TABLE 6.—*Percentage distribution of State highway mileage surfaced in the years from 1923 to 1931*

Year	Gravel	Water-bound macadam	Bituminous macadam	Bituminous concrete	Portland cement concrete	Brick	Total
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1923	54.22	7.02	6.84	3.53	26.77	1.62	100.00
1924	52.56	8.52	5.79	3.98	28.19	.96	100.00
1925	56.10	6.41	5.61	3.17	27.89	.82	100.00
1926	62.96	5.58	5.81	2.36	22.59	.70	100.00
1927	56.88	8.72	5.65	2.56	25.84	.35	100.00
1928	52.60	4.89	9.62	2.90	29.43	.56	100.00
1929	58.16	6.55	4.79	2.22	27.88	.40	100.00
1930	55.73	4.99	4.65	2.54	31.50	.59	100.00
1931	59.56	4.21	5.61	2.17	27.98	.47	100.00

The conversion of these percentages into the yardage in the composite mile assignable to each type is shown in table 7. It will be observed that the surfaced yardage in the composite mile varies from year to year, as indicated in a preceding paragraph.

TABLE 7.—*Square yards per surfaced mile assignable to each type of surfacing*

Year	Total	Gravel	Water-bound macadam	Bituminous macadam	Bituminous concrete	Portland cement concrete	Brick
	Sq. yd.	*Sq. yd.*	*Sq. yd.*	*Sq. yd.*	*Sq. yd.*	*Sq. yd.*	*Sq. yd.*
1923	10,325	5,598	725	706	365	2,764	167
1924	10,501	5,519	895	608	418	2,960	101
1925	10,677	5,990	684	599	338	2,978	88
1926	10,853	6,833	606	630	256	2,452	76
1927	11,029	6,273	962	623	282	2,850	39
1928	11,205	5,894	548	1,078	325	3,297	63
1929	11,381	6,619	745	545	253	3,173	46
1930	11,557	6,441	577	537	291	3,640	68
1931	11,733	6,988	494	658	255	3,283	55

For reasons previously explained, gravel and water-bound macadam have been converted into an equivalent yardage of excavation. Similarly, bituminous macadam, bituminous concrete, and brick have been converted into an equivalent yardage of concrete surfacing. The details of these conversions are shown in table 8.

TABLE 8.—*Equivalent quantities of excavation and pavement*

Year	Equivalent excavation in cubic yards		Equivalent portland cement concrete pavement in square yards			
	Gravel (0.6871)[1]	Water-bound macadam (1.3818)[1]	Bituminous macadam (0.6423)[1]	Bituminous concrete (0.8405)[1]	Brick (1.3581)[1]	Portland cement concrete
1923	3,846	1,002	453	307	227	2,764
1924	3,792	1,237	391	351	137	2,960
1925	4,116	945	385	284	120	2,978
1926	4,695	837	405	215	103	2,452
1927	4,310	1,329	400	237	53	2,850
1928	4,050	757	692	273	86	3,297
1929	4,548	1,029	350	213	62	3,173
1930	4,426	797	345	247	92	3,640
1931	4,801	683	423	214	75	3,283

[1] Conversion factor.

The adjustments were made by applying the ratios of the base prices as used in the computation of the price trend. These ratios were as follows:

Gravel to excavation:

$$\frac{0.235}{0.342}=0.6871$$

Water-bound macadam to crushed stone (2,350 cubic yards of crushed stone per mile):

$$\frac{2,350}{10,560}=0.2225$$

Crushed stone to excavation:

$$\frac{2.124}{0.342}=6.2105$$

Water-bound macadam to excavation:

$$6.2105\times 0.2225=1.3818$$

Bituminous macadam to portland cement concrete:

$$\frac{1.426}{2.220}=0.6423$$

Bituminous concrete to portland cement concrete:

$$\frac{1.866}{2.220}=0.8405$$

Brick to portland cement concrete:

$$\frac{3.015}{2.220}=1.3581$$

As the result of these operations, the final quantities per surfaced mile are as shown in table 9.

TABLE 9.—*Final quantities per surfaced mile*

Year	Excavation in cubic yards				Pavement in square yards				Structures		
	From excavation	From gravel	From water-bound macadam	Total excavation	From bituminous macadam	From bituminous concrete	From brick	From Portland cement concrete	Total pavement	Reinforcing steel	Structural steel	Structural concrete
										Pounds	*Pounds*	*Cubic yards*
1923	8,068	3,846	1,002	12,916	453	307	227	2,764	3,751	9,270	2,297	53
1924	8,364	3,792	1,237	13,393	391	351	137	2,960	3,839	12,374	2,258	67
1925	9,238	4,116	945	14,299	385	284	120	2,978	3,767	13,581	4,718	64
1926	11,068	4,695	837	16,600	405	215	103	2,452	3,175	14,070	3,629	68
1927	10,960	4,310	1,329	16,599	400	237	53	2,850	3,540	12,773	3,301	64
1928	12,545	4,050	757	17,352	692	273	86	3,297	4,348	17,075	4,953	65
1929	17,028	4,548	1,029	22,605	350	213	62	3,173	3,798	22,503	5,024	81
1930	18,946	4,426	797	24,169	345	247	92	3,640	4,324	26,852	7,756	122
1931	22,361	4,801	683	27,845	423	214	75	3,283	3,995	30,751	12,216	141

TABLE 10.—*Adjusted average bid prices for common excavation* [1]

State	Base quantities	National weighting	1922		1923		1924		1925		1926		1927		1928		1929		1930	
			Bid price	W	Bid price	W	Bid price	W	Bid price	W	Bid price	W	Bid price	W	Bid price	W	Bid price	W	Bid price	W
	Cubic yards																			
New England	8,672,136	0.022																		
Maine	1,237,391	.003	$1.23	0.0037	$1.52	0.0046	$1.23	0.0037	$1.18	0.0035	$1.07	0.0032	$1.00	0.0030	$1.02	0.0031	$0.81	0.0024	$0.65	0.001
New Hampshire	620,590	.002	1.02	.0020	1.47	.0029	1.32	.0026	.99	.0020	1.17	.0023	.93	.0019	.91	.0018	.65	.0013	.51	.001
Vermont	1,371,532	.003	1.01	.0030	1.28	.0038	1.29	.0039	1.19	.0036	1.20	.0036	1.04	.0031	.93	.0028	.73	.0022	.69	.002
Massachusetts	2,703,415	.007	.92	.0064	1.32	.0092	1.19	.0083	1.12	.0078	.81	.0057	.72	.0050	.77	.0054	.77	.0054	.48	.003
Rhode Island	1,136,757	.003	.95	.0028	1.28	.0038	1.00	.0030	.92	.0028	.90	.0027	.84	.0025	.50	.0015	.42	.0013	.42	.001
Connecticut	1,602,451	.004	.93	.0037	1.25	.0050	1.21	.0048	1.17	.0047	.75	.0030	.85	.0034	.89	.0036	.73	.0029	.59	.002
Middle Atlantic	46,955,531	.118																		
New York	20,083,395	.051	.91	.0309	.87	.0296	.92	.0313	.84	.0286	.93	.0316	.66	.0224	.64	.0218	.53	.0180	.54	.018
New Jersey	4,515,264	.011	1.36	.0150	1.52	.0167	1.54	.0169	1.22	.0134	1.04	.0114	.82	.0090	.84	.0092	.56	.0062	.65	.007
Pennsylvania	22,356,872	.056	1.10	.0407	1.38	.0511	1.33	.0492	1.16	.0429	.88	.0326	.99	.0366	.79	.0292	.75	.0277	.77	.028
East North Central	39,897,012	.100																		
Ohio	10,498,485	.026	.65	.0169	.53	.0138	.59	.0153	.56	.0146	.56	.0146	.46	.0126	.49	.0127	.48	.0125	.45	.011
Indiana	4,889,953	.012	.52	.0062	.50	.0060	.33	.0040	.49	.0059	.40	.0048	.43	.0052	.41	.0049	.40	.0048	.38	.004
Illinois	5,731,257	.014	.43	.0060	.48	.0067	.41	.0057	.44	.0062	.40	.0056	.45	.0063	.41	.0057	.50	.0070	.34	.004
Michigan	6,962,724	.018	.47	.0085	.59	.0106	.51	.0092	.46	.0083	.45	.0081	.44	.0079	.42	.0076	.38	.0068	.30	.005
Wisconsin	11,814,593	.030	.40	.0120	.57	.0171	.42	.0126	.46	.0138	.42	.0126	.48	.0144	.44	.0132	.40	.0120	.33	.009
West North Central	103,605,082	.261																		
Minnesota	23,505,790	.059	.31	.0183	.31	.0183	.26	.0153	.25	.0147	.26	.0153	.27	.0159	.25	.0147	.25	.0147	.31	.018
Iowa	9,209,728	.023	.27	.0062	.29	.0067	.29	.0067	.23	.0053	.28	.0064	.43	.0099	.36	.0083	.32	.0074	.40	.009
Missouri	10,298,517	.026	.34	.0088	.33	.0086	.35	.0091	.38	.0099	.30	.0078	.32	.0083	.31	.0081	.36	.0094	.34	.008
North Dakota	16,816,188	.042	.27	.0113	.31	.0130	.27	.0113	.28	.0118	.31	.0130	.30	.0126	.29	.0122	.25	.0105	.23	.009
South Dakota	13,023,156	.033	.25	.0082	.28	.0092	.25	.0082	.21	.0069	.21	.0069	.20	.0066	.21	.0069	.19	.0063	.18	.005
Nebraska	17,716,058	.045	.25	.0112	.46	.0207	.24	.0108	.23	.0103	.20	.0090	.19	.0085	.18	.0081	.19	.0085	.16	.007
Kansas	13,035,645	.033	.42	.0139	.39	.0129	.39	.0129	.33	.0109	.30	.0099	.27	.0089	.26	.0086	.27	.0089	.22	.007
South Atlantic	25,570,554	.064																		
Delaware	563,300	.001	.83	.0008	1.21	.0012	1.27	.0013	1.53	.0015	.75	.0007	.64	.0006	.59	.0006	.64	.0006	.63	.000
Maryland	324,700	.001	.72	.0007	.89	.0009	.85	.0008	1.02	.0010	.74	.0007	.63	.0006	.67	.0007	.57	.0006	.30	.000
Virginia	3,000,533	.007	.49	.0024	.57	.0028	.44	.0022	.65	.0032	.50	.0025	.41	.0020	.40	.0020	.71	.0035	.43	.002
West Virginia	5,669,135	.014	.71	.0064	.67	.0060	.49	.0044	.69	.0062	.56	.0050	.50	.0045	.51	.0046	.50	.0045	.48	.004
North Carolina	3,896,910	.010	.81	.0081	.48	.0048	.53	.0053	.55	.0055	.49	.0049	.38	.0038	.33	.0033	.32	.0032	.29	.002
South Carolina	5,211,802	.013	.27	.0035	.29	.0038	.32	.0042	.39	.0051	.31	.0040	.33	.0043	.31	.0040	.29	.0038	.32	.004
Georgia	5,024,754	.013	.23	.0030	.31	.0040	.34	.0044	.36	.0047	.35	.0045	.29	.0038	.31	.0040	.30	.0039	.29	.003
Florida	1,879,420	.005	.36	.0018	.58	.0029	.53	.0026	.56	.0028	.48	.0024	.37	.0018	.33	.0016	.32	.0016	.31	.001
East South Central	37,814,702	.095																		
Kentucky	13,903,434	.035	.41	.0143	.50	.0175	.42	.0147	.36	.0126	.34	.0119	.38	.0133	.41	.0143	.36	.0126	.38	.013
Tennessee	6,486,454	.016	.37	.0059	.30	.0048	.44	.0070	.36	.0058	.38	.0061	.33	.0053	.44	.0070	.33	.0053	.34	.005
Alabama	12,596,256	.032	.28	.0090	.32	.0102	.30	.0096	.28	.0090	.31	.0099	.29	.0093	.27	.0086	.27	.0086	.25	.008
Mississippi	4,828,558	.012	.26	.0031	.29	.0035	.28	.0034	.33	.0040	.30	.0036	.28	.0034	.28	.0034	.26	.0031	.26	.003
West South Central	36,011,743	.091																		
Arkansas	6,605,424	.017	.21	.0036	.62	.0105	.48	.0082	.23	.0039	.26	.0044	.29	.0049	.27	.0046	.25	.0042	.22	.003
Louisiana	5,662,812	.014	.28	.0039	.24	.0034	.36	.0050	.19	.0027	.52	.0073	.18	.0025	.23	.0032	.27	.0038	.26	.003
Oklahoma	2,055,775	.005	.26	.0013	.27	.0013	.27	.0013	.16	.0008	.26	.0013	.24	.0012	.24	.0012	.23	.0011	.23	.001
Texas	21,687,732	.055	.16	.0088	.17	.0093	.19	.0104	.19	.0104	.21	.0115	.18	.0099	.17	.0093	.18	.0099	.19	.010
Mountain	59,477,256	.149																		
Montana	23,225,283	.056	.24	.0134	.37	.0207	.28	.0157	.24	.0134	.25	.0140	.29	.0162	.25	.0140	.24	.0134	.29	.016
Idaho	3,602,469	.009	.28	.0025	.33	.0030	.33	.0030	.28	.0025	.25	.0022	.32	.0029	.32	.0029	.29	.0026	.25	.002
Wyoming	9,009,288	.023	.20	.0046	.23	.0053	.29	.0067	.23	.0053	.20	.0046	.21	.0048	.23	.0053	.21	.0048	.21	.004
Colorado	7,355,863	.019	.27	.0051	.33	.0063	.39	.0074	.29	.0055	.32	.0061	.27	.0051	.31	.0059	.33	.0063	.20	.003
New Mexico	3,839,591	.010	.28	.0028	.29	.0029	.31	.0031	.40	.0040	.27	.0027	.19	.0019	.23	.0023	.20	.0020	.21	.002
Arizona	3,092,928	.008	.95	.0047	1.10	.0055	1.11	.0055	.79	.0039	.35	.0017	.70	.0035	.50	.0025	.41	.0020	.67	.003
Utah	3,570,442	.009	.39	.0035	.45	.0040	.42	.0038	.25	.0022	.31	.0028	.34	.0031	.36	.0032	.33	.0030	.26	.002
Nevada	5,781,392	.015	.71	.0071	.74	.0074	.74	.0074	.53	.0053	.44	.0044	.50	.0050	.26	.0026	.35	.0035	.34	.003
Pacific	39,670,604	.100																		
Washington	11,907,165	.030	.45	.0135	.48	.0144	.41	.0123	.38	.0114	.29	.0087	.36	.0108	.38	.0114	.34	.0102	.25	.007
Oregon	8,377,200	.021	.37	.0078	.37	.0078	.35	.0073	.42	.0088	.37	.0078	.27	.0057	.40	.0084	.34	.0071	.27	.005
California	19,386,239	.049	.75	.0247	1.03	.0340	.82	.0271	.50	.0165	.55	.0181	.57	.0188	.51	.0168	.43	.0142	.45	.014
United States	397,674,620	1.000		.4020		.4685		.4289		.3859		.3639		.3524		.3371		.3156		.303

[1] Unclassified excavation predominates in New York, Pennsylvania, West Virginia, Virginia, Arizona, Nevada, and California, and the bid prices shown for these States are for unclassified excavation. In order to convert these prices into equivalent common excavation, the weighting values were correspondingly adjusted. The resultant weighting values for these States as used in this table are as follows: New York, 0.034; Pennsylvania, 0.037; Virginia, 0.005; West Virginia, 0.009; Arizona, 0.005; Nevada, 0.010; California, 0.033.

BID PRICES ON FEDERAL-AID WORK USED IN COMPUTATION OF PRICE INDEX

The final step in the preparation of index numbers involves the application of prices to the quantities selected as a base. All prices are from the Bureau's records of bid prices, which prior to October 1931 cover Federal-aid construction only. Since that date, prices on State work have been available and are included in the averages.

Throughout the highway construction field variations in the controlling specifications cause prices to vary considerably from State to State. In a general way prices of materials and labor are higher in the North than they are in the South; but as work can be carried on throughout the whole of the year in the South while it can be satisfactorily handled in the North only during the warmer months, the regional volume of work awarded is not uniform from month to month. Neither does the influence of these two very general regions on the amount awarded remain constant.

To avoid the effect of these and of other causes which produce variations in a given region during a given time

TABLE 10.—*Adjusted average bid prices for common excavation* Continued

State	Base quantities	National weighting	1931 First quarter Bid price	1931 First quarter W	1931 Second quarter Bid price	1931 Second quarter W	1931 Third quarter Bid price	1931 Third quarter W	1931 Fourth quarter Bid price	1931 Fourth quarter W	1932 First quarter Bid price	1932 First quarter W	1932 Second quarter Bid price	1932 Second quarter W	1932 Third quarter Bid price	1932 Third quarter W	1932 Fourth quarter Bid price	1932 Fourth quarter W	1933 First quarter Bid price	1933 First quarter W
	Cubic yards																			
New England	8,672,136	0.022																		
Maine	1,237,391	.003	$0.35	0.0010	$0.55	0.0016	$0.18	0.0014	$0.35	0.0010	$0.31	0.0009	$0.43	0.0013	$0.41	0.0012	$0.41	0.0012	$0.41	0.0012
New Hampshire	620,590	.002	.48	.0010	.33	.0007	.46	.0008	.29	.0006	.26	.0005	.32	.0006	.25	.0005	.29	.0006	.46	.0009
Vermont	1,371,532	.003	.57	.0017	.47	.0014	.41	.0013	.32	.0010	.28	.0008	.33	.0010	.20	.0006	.21	.0006	.21	.0006
Massachusetts	2,703,415	.007	.49	.0034	.43	.0030	.30	.0021	.27	.0019	.24	.0017	.20	.0014	.20	.0014	.19	.0013	.26	.0018
Rhode Island	1,136,757	.003	.35	.0010	.25	.0007	.25	.0007	.22	.0007	.20	.0006	.23	.0007	.13	.0001	.15	.0004	.33	.0010
Connecticut	1,602,451	.004	.52	.0021	.50	.0020	.70	.0028	.40	.0016	.36	.0014	.42	.0017	.25	.0010	.31	.0012	.38	.0015
Middle Atlantic	46,955,531	.118																		
New York	20,083,395	.051	.40	.0136	.38	.0129	.35	.0119	.50	.0170	.29	.0099	.24	.0082	.23	.0078	.26	.0088	.30	.0102
New Jersey	4,515,264	.011	.65	.0071	.23	.0025	.22	.0024	.22	.0024	.20	.0022	.16	.0018	.27	.0030	.20	.0022	.20	.0022
Pennsylvania	22,356,872	.056	1.75	.0647	1.39	.0514	.71	.0263	.30	.0111	.45	.0166	.35	.0129	.34	.0126	.36	.0133	.37	.0137
East North Central	39,897,012	.100																		
Ohio	10,498,485	.026	.30	.0078	.32	.0083	.34	.0088	.26	.0068	.35	.0091	.17	.0044	.17	.0044	.16	.0042	.18	.0047
Indiana	4,889,953	.012	.29	.0035	.30	.0036	.27	.0032	.24	.0029	.22	.0026	.21	.0025	.20	.0024	.21	.0025	.22	.0026
Illinois	5,731,257	.014	.32	.0045	.30	.0042	.28	.0039	.21	.0029	.23	.0032	.21	.0029	.24	.0034	.23	.0032	.22	.0031
Michigan	6,962,724	.018	.27	.0049	.25	.0045	.20	.0036	.25	.0045	.21	.0038	.21	.0038	.19	.0034	.21	.0038	.21	.0038
Wisconsin	11,814,593	.030	.31	.0093	.30	.0090	.26	.0078	.90	.0270	.31	.0093	.22	.0066	.22	.0066	.18	.0054	.27	.0081
West North Central	103,605,082	.261																		
Minnesota	23,505,790	.059	.18	.0106	.20	.0118	.46	.0271	.36	.0212	.17	.0100	.15	.0088	.12	.0071	.15	.0088	.14	.0083
Iowa	9,209,728	.023	.33	.0076	.31	.0071	.31	.0071	.20	.0046	.14	.0032	.12	.0028	.14	.0032	.10	.0023	.10	.0023
Missouri	10,298,517	.026	.35	.0091	.28	.0073	.24	.0062	.24	.0062	.19	.0049	.20	.0052	.20	.0052	.22	.0057	.20	.0052
North Dakota	16,816,188	.042	.20	.0084	.21	.0088	.20	.0084	.15	.0063	.13	.0055	.13	.0055	.15	.0063	.16	.0067	.15	.0063
South Dakota	13,023,156	.033	.17	.0056	.19	.0063	.19	.0063	.11	.0036	.12	.0040	.11	.0036	.13	.0043	.15	.0049	.13	.0043
Nebraska	17,716,058	.045	.16	.0072	.18	.0081	.19	.0085	.16	.0072	.15	.0067	.14	.0063	.14	.0063	.15	.0067	.11	.0063
Kansas	13,035,645	.033	.20	.0066	.20	.0066	.20	.0066	.16	.0053	.14	.0046	.14	.0046	.15	.0049	.13	.0043	.15	.0049
South Atlantic	25,570,554	.064																		
Delaware	563,300	.001	.44	.0004	.52	.0005	.66	.0007	.40	.0004	.29	.0003	.21	.0002	.17	.0002	.40	.0001	.40	.0004
Maryland	324,700	.001	.52	.0005	.43	.0004	.54	.0005	.23	.0002	.17	.0002	.20	.0002	.19	.0002	.22	.0002	.28	.0003
Virginia	3,000,533	.007	.45	.0022	.58	.0029	.46	.0023	.31	.0015	.21	.0010	.19	.0009	.31	.0015	.30	.0015	.28	.0014
West Virginia	5,669,135	.014	.36	.0032	.36	.0032	.33	.0030	.23	.0021	.39	.0035	.25	.0022	.24	.0022	.31	.0028	.28	.0025
North Carolina	3,896,910	.010	.37	.0037	.30	.0030	.24	.0024	.37	.0037	.35	.0035	.18	.0018	.24	.0024	.32	.0032	.30	.0030
South Carolina	5,211,802	.013	.36	.0047	.33	.0043	.42	.0055	.24	.0031	.17	.0022	.27	.0035	.22	.0029	.25	.0032	.27	.0035
Georgia	5,024,754	.013	.27	.0035	.25	.0032	.24	.0031	.20	.0026	.13	.0017	.17	.0022	.12	.0016	.22	.0029	.30	.0039
Florida	1,879,420	.005	.33	.0016	.31	.0015	1.00	.0050	.68	.0034	.22	.0011	.26	.0013	.33	.0016	.29	.0014	.39	.0019
East South Central	37,814,702	.095																		
Kentucky	13,903,434	.035	.29	.0101	.27	.0094	.26	.0091	.17	.0059	.14	.0049	.22	.0077	.19	.0066	.17	.0059	.18	.0063
Tennessee	6,486,454	.016	.41	.0066	.30	.0048	.50	.0080	.26	.0042	.21	.0034	.19	.0030	.26	.0042	.23	.0037	.30	.0048
Alabama	12,596,256	.032	.20	.0064	.24	.0077	.29	.0093	.18	.0058	.15	.0048	.16	.0051	.26	.0083	.20	.0064	.25	.0080
Mississippi	4,828,558	.012	.23	.0028	.23	.0028	.24	.0029	.19	.0023	.16	.0019	.15	.0018	.18	.0022	.21	.0025	.21	.0025
West South Central	36,011,743	.091																		
Arkansas	6,605,424	.017	.24	.0041	.24	.0041	.15	.0025	.13	.0022	.11	.0019	.11	.0019	.14	.0024	.25	.0042	.18	.0031
Louisiana	5,662,812	.014	.25	.0035	.22	.0031	.18	.0025	.18	.0025	.15	.0021	.17	.0024	.21	.0029	.17	.0024	.21	.0029
Oklahoma	2,055,775	.005	.22	.0011	.18	.0009	.18	.0009	.14	.0007	.12	.0006	.10	.0005	.13	.0006	.13	.0006	.14	.0007
Texas	21,687,732	.055	.18	.0099	.14	.0077	.14	.0077	.12	.0066	.10	.0055	.10	.0055	.12	.0066	.14	.0077	.21	.0115
Mountain	59,477,256	.149																		
Montana	23,225,283	.056	.22	.0123	.31	.0174	.21	.0118	.15	.0084	.12	.0067	.14	.0078	.31	.0174	.26	.0146	.20	.0112
Idaho	3,602,469	.009	.30	.0018	.25	.0022	.27	.0024	.16	.0014	.19	.0017	.23	.0021	.26	.0023	.20	.0018	.36	.0032
Wyoming	9,009,288	.023	.22	.0051	.20	.0046	.18	.0041	.16	.0037	.15	.0034	.16	.0037	.17	.0039	.17	.0039	.16	.0037
Colorado	7,355,863	.019	.15	.0028	.17	.0032	.18	.0031	.33	.0063	.14	.0027	.18	.0031	.20	.0038	.16	.0030	.14	.0027
New Mexico	3,839,591	.010	.29	.0029	.27	.0027	.25	.0025	.14	.0014	.11	.0011	.17	.0017	.26	.0026	.19	.0019	.20	.0020
Arizona	3,092,928	.008	.27	.0013	.52	.0026	.59	.0029	.48	.0024	.31	.0015	.35	.0017	.51	.0025	.17	.0023	.61	.0032
Utah	3,570,442	.009	.23	.0021	.21	.0019	.42	.0038	.17	.0015	.15	.0013	.14	.0013	.13	.0012	.15	.0013	.21	.0019
Nevada	5,781,392	.015	.24	.0024	.62	.0062	.33	.0033	.16	.0016	.20	.0020	.23	.0023	.23	.0023	.32	.0032	.35	.0035
Pacific	39,670,604	.100																		
Washington	11,907,165	.030	.22	.0066	.28	.0084	.26	.0078	.20	.0060	.23	.0069	.21	.0063	.27	.0081	.19	.0057	.14	.0042
Oregon	8,377,200	.021	.20	.0042	.46	.0097	.27	.0057	.22	.0046	.24	.0050	.24	.0050	.25	.0052	.20	.0042	.31	.0071
California	19,386,239	.049	.31	.0102	.30	.0099	.28	.0092	.24	.0079	.25	.0082	.27	.0089	.30	.0099	.35	.0115	.24	.0079
United States	397,674,620	1.000		.2967		.2901		.2695		.2282		.1806		.1710		.1916		.1905		.2003

the influence of each State has been weighted. To obtain these weightings the whole amount of work performed on Federal-aid projects on each of the selected items during the base period was ascertained for each State. This amount was then reduced to a percentage assignable to each of the States where work of this kind was done. These percentages are used as indicating the average influence of the various States in determining the average price at which the item in question is produced. For any period the national average price is, then, the sum of the products obtained by multiplying each average State price (as determined from the Bureau's records of bid prices) by the corresponding State influence rating.

It seldom happens that during a given quarter every State awards enough highway work so that all States report prices on every one of the items used in this index. There are, of course, several ways in which this deficiency can be adjusted. These need no discussion here. Suffice it to say that for this index the practice has been adopted of using estimated prices based on regional experience wherever prices are missing. Thus, if of several States in a geographic division there is one for which no new price is of record, but those

reporting show an average drop in price of 3 percent, the missing price is supplied at 97 percent of the price shown for the preceding quarter. If no prices are received from a geographic division or if too small percentage of the States report new prices, the last reported prices are used as the current prices.

Table 10 shows the prices on common excavation as calculated in this way. The columns headed "W" in this table give for each year the product of the average bid price in a given State times the national weighting of that State. As stated previously, the sum of these products, given at the foot of the column, is the average bid price for the year.

Base quantities and base prices are shown in table 11. The prices used in computing the base are the arithmetical average of the prices shown by years for the base period.

TABLE 11.—*Average quantities and prices for the years 1925 to 1929, used as base for computation of indexes*

Year	Excavation		Pavement		Reinforcing steel		Structural steel		Structural concrete	
	Quantity	Price per cubic yard	Quantity	Price per square yard	Quantity	Price per pound	Quantity	Price per pound	Quantity	Price per cubic yard
	Cu. yds.		*Sq. yds.*		*Lbs.*		*Lbs.*		*Cu. yds.*	
1925	14,299	$0.386	3,767	$2.360	13,581	$0.0565	4,718	$0.0667	64	$22.534
1926	16,600	.364	3,175	2.286	14,070	.0534	3,629	.0736	68	22.760
1927	16,599	.352	3,540	2.291	12,773	.0510	3,301	.0707	64	22.647
1928	17,352	.337	4,348	2.096	17,075	.0492	4,953	.0671	65	21.216
1929	22,605	.316	3,798	2.055	22,503	.0481	5,024	.0591	81	21.582
Total	87,455	1.755	18,628	11.088	80,002	.2582	21,625	.3372	342	110.739
Average	17,491	.351	3,726	2.218	16,000	.0516	4,325	.0674	68	22.148

THE PRICE INDEX

The final computation of the price index is shown in table 12. The composite mile on which the price index is based is composed of the average quantities of excavation, pavement, reinforcing steel, structural steel, and structural concrete determined for the base years 1925 to 1929, and shown in table 11. In table 12 the average bid price for each item in each year is shown. Under the heading "Amount" is given the cost of the average (base) quantity of each item at the prevailing bid price of the year. The subindexes are proportional to these amounts, index value 100 being based on the average bid prices for the base years, as given in table 11. Similarly, under the heading "Composite mile" the total amounts and the corresponding indexes are given for each year. The variation of these price indexes is shown in graphical form in figure 1.

THE USAGE INDEX

A usage index differs from a price index in that it shows the effect of changing usage rather than changing price. This is done by applying fixed prices—in this case, those of the base period—to the varying quantities of the several items of which the base is composed, the result being to show how cost would have varied from year to year on account of changing usage if prices had remained constant.

In the highway field usage has changed a great deal since the first year (1923) for which figures are available. There probably has been some change in pavement thickness during this period, but as this change is believed to have been small and as no analysis of the amount of it has ever been attempted it is included in none of the usage calculations.

TABLE 12.—*Price trend in highway construction*

Year	Excavation [1] (17,491 cubic yards)			Pavement [2] (3,726 square yards)			Structures								Composite mile	
							Reinforcing steel (16,000 pounds)		Structural steel (4,325 pounds)		Structural concrete (68 cubic yards)		Combined			
	Bid price [3]	Amount	Subindex	Bid price [3]	Amount	Subindex	Bid price [3]	Amount	Bid price [3]	Amount	Bid price [3]	Amount	Amount	Subindex	Total amount	Index
Base period, 1925 to 1929	$0.35	$6,139	100.0	$2.22	$8,264	100.0	$0.052	$826	$0.067	$291	$22.15	$1,506	$2,623	100.0	$17,026	100.0
1922	.40	7,031	114.5	2.28	8,488	102.7	.050	800	.074	321	20.18	1,372	2,493	95.1	18,012	105.8
1923	.47	8,186	133.3	2.43	9,047	109.5	.057	920	.078	338	23.37	1,589	2,847	108.6	20,080	117.9
1924	.43	7,504	122.2	2.40	8,950	108.3	.057	920	.077	333	22.91	1,558	2,811	107.2	19,265	113.1
1925	.39	6,751	110.0	2.36	8,793	106.4	.056	904	.067	288	22.53	1,532	2,724	103.9	18,268	107.3
1926	.36	6,367	103.7	2.29	8,518	103.1	.053	854	.074	318	22.76	1,548	2,720	103.7	17,605	103.4
1927	.35	6,157	100.3	2.29	8,536	103.3	.051	816	.071	306	22.65	1,540	2,662	101.5	17,355	101.9
1928	.34	5,894	96.0	2.10	7,810	94.5	.049	787	.067	290	21.22	1,443	2,520	96.1	16,224	95.3
1929	.32	5,527	90.0	2.05	7,657	92.7	.048	770	.059	256	21.58	1,468	2,494	95.0	15,678	92.1
1930	.30	5,300	86.3	1.86	6,949	84.1	.045	715	.061	264	20.98	1,365	2,344	89.4	14,593	85.7
1931																
First quarter	.30	5,195	84.6	1.79	6,666	80.7	.042	672	.055	240	18.90	1,285	2,197	83.7	14,058	82.6
Second quarter	.29	5,072	82.6	1.77	6,580	79.6	.041	658	.051	219	18.48	1,257	2,134	81.3	13,786	81.0
Third quarter	.27	4,705	76.6	1.59	5,924	71.7	.040	634	.052	224	17.49	1,189	2,047	78.0	12,676	74.4
Fourth quarter	.23	3,988	65.0	1.56	5,809	70.3	.037	594	.056	241	17.22	1,171	2,009	76.6	11,806	69.3
1932																
First quarter	.18	3,166	51.6	1.52	5,656	68.4	.036	571	.049	211	15.22	1,035	1,817	69.3	10,639	62.5
Second quarter	.17	2,991	48.7	1.47	5,481	66.3	.034	538	.045	197	14.98	1,019	1,754	66.8	10,226	60.1
Third quarter	.19	3,358	54.7	1.35	5,045	61.0	.033	526	.043	184	14.82	1,008	1,718	65.5	10,121	59.4
Fourth quarter	.19	3,323	54.1	1.44	5,377	65.1	.033	528	.048	208	16.28	1,107	1,843	70.3	10,543	61.9
1933																
First quarter	.20	3,498	57.0	1.49	5,552	67.2	.032	506	.043	187	15.44	1,050	1,743	66.4	10,793	63.4

[1] Common excavation plus other items expressed as equivalent common excavation. (See p. 85 and table 8.)
[2] Portland cement concrete plus other items expressed as equivalent portland cement concrete. (See p. 85 and table 8.)
[3] Indexes and totals were calculated with the bid prices carried to 1 more decimal place than that to which they are shown in this table.

TABLE 13.—*Usage trend in highway construction*

Year	Excavation [1] ($0.351 per cubic yard)			Pavement [2] ($2.218 per square yard)			Structural: Reinforcing steel ($0.0516 per pound)		Structural: Structural steel ($0.0674 per pound)		Structural: Structural concrete ($22.148 per cubic yard)		Structural: Combined		Composite mile	
	Quantities	Amount	Subindex	Quantities	Amount	Subindex	Quantities	Amount	Quantities	Amount	Quantities	Amount	Amount	Subindex	Total amount	Index
Base period, 1925 to 1929	17,491	$6,139	100.0	3,726	$8,264	100.0	16,000	$826	4,325	$291	68	$1,506	$2,623	100.0	$17,026	100.0
1923	12,916	4,533	73.8	3,751	8,320	100.7	9,270	478	2,297	155	53	1,174	1,807	68.9	14,660	86.1
1924	13,393	4,701	76.6	3,839	8,515	103.0	12,374	638	2,258	152	67	1,484	2,274	86.7	15,490	91.0
1925	14,299	5,019	81.7	3,767	8,355	101.1	13,581	701	4,718	318	64	1,417	2,436	92.9	15,810	92.9
1926	16,600	5,827	94.9	3,175	7,042	85.2	14,070	726	3,629	245	68	1,506	2,477	94.4	15,346	90.1
1927	16,599	5,826	94.9	3,540	7,852	95.0	12,773	659	3,301	222	64	1,417	2,298	87.6	15,976	93.8
1928	17,352	6,091	99.2	4,348	9,644	116.7	17,075	881	4,953	334	65	1,440	2,655	101.2	18,390	108.0
1929	22,605	7,934	129.2	3,798	8,424	101.9	22,503	1,161	5,024	339	81	1,794	3,294	125.6	19,652	115.4
1930	24,169	8,483	138.2	4,324	9,591	116.0	26,852	1,386	7,750	522	122	2,702	4,610	175.7	22,684	133.2
1931	27,845	9,774	159.2	3,995	8,861	107.2	30,751	1,587	12,216	823	141	3,123	5,533	210.9	24,168	141.9

[1] Common excavation plus other items expressed as equivalent common excavation. (See p. 85 and table 8.)
[2] Portland cement concrete plus other pavement items expressed as equivalent Portland cement concrete. (See p. 85 and table 8.)

As noted in a previous paragraph, the average width of the surfacing has increased rather steadily at the rate of about 0.3 foot per year. This has affected excavation quantities and structural quantities, as well as surfacing quantities.

The amount of excavation handled per mile of highway constructed has been further increased by the use of wider shoulders and lighter side slopes. Fifteen years ago 16-foot pavement and 3-foot shoulders, giving an embankment width of 22 feet, were common. Today 20-foot pavement and 6-foot shoulders, giving an embankment width of 32 feet are almost as common, with wider shoulders not infrequently used.

Finally, as compared to the conditions prevailing even 10 years ago, there is a marked tendency to hold to improved standards of both alinement and grade even where this policy requires a great deal of heavy grading. Again the necessary result is increased quantities. In the light of these conditions, the fact that the excavation handled per mile of highway built has more than doubled during the period covered by this index is not surprising.

In the structural field the situation is similar. Pavements have increased in width, and this has forced a corresponding increase in the width of bridges and in the length of culverts. Formerly both were governed quite largely by the width of the surfacing. In more recent years there has been a tendency to make bridges wider than the surfaced roadway. There has also been a marked tendency to make culverts at least as long as the width of the embankment (surfacing plus shoulders). As a result of these changes the structural quantities used per mile of highway constructed have also more than doubled.

TABLE 14.—*Cost trend in highway construction*

Year	Excavation [1]				Pavement [2]			
	Bid price [3]	Quantity	Amount	Subindex	Bid price [3]	Quantity	Amount	Subindex
		Cubic yards				*Square yards*		
Base period, 1925 to 1929	$0.35	17,491	$6,139	100.0	$2.22	3,726	$8,264	100.0
1923	.47	12,916	6,045	98.5	2.43	3,751	9,107	110.2
1924	.43	13,393	5,746	93.6	2.40	3,839	9,221	111.6
1925	.39	14,299	5,519	89.9	2.36	3,767	8,890	107.6
1926	.36	16,600	6,042	98.4	2.29	3,175	7,258	87.8
1927	.35	16,599	5,843	95.2	2.29	3,540	8,110	98.1
1928	.34	17,352	5,848	95.2	2.10	4,348	9,113	110.3
1929	.32	22,605	7,143	116.4	2.05	3,798	7,805	94.4
1930	.30	24,169	7,323	119.3	1.86	4,324	8,064	97.6
1931	.27	27,845	7,546	122.9	1.68	3,995	6,696	81.0

Year	Structures: Reinforcing steel			Structural steel			Structural concrete			Combined		Composite mile	
	Bid price [3]	Quantity	Amount	Bid price [3]	Quantity	Amount	Bid price [3]	Quantity	Amount	Amount	Subindex	Amount	Index
		Pounds			*Pounds*			*Cu. yds.*					
Base period 1925 to 1929	$0.052	16,000	$826	$0.067	4,325	$291	$22.15	68	$1,506	$2,623	100.0	$17,026	100.0
1923	.057	9,270	533	.078	2,297	180	23.37	53	1,239	1,952	74.4	17,104	100.5
1924	.057	12,374	714	.077	2,258	174	22.91	67	1,535	2,420	92.3	17,387	102.1
1925	.056	13,581	767	.067	4,718	315	22.53	64	1,442	2,524	96.2	16,933	99.5
1926	.053	14,070	754	.074	3,629	267	22.76	68	1,548	2,566	97.8	15,866	93.2
1927	.051	12,773	654	.071	3,301	233	22.65	64	1,449	2,333	89.0	16,286	95.7
1928	.049	17,075	840	.067	4,953	332	21.22	65	1,379	2,551	97.3	17,512	102.9
1929	.048	22,503	1,082	.059	5,024	297	21.58	81	1,748	3,127	119.2	18,075	106.2
1930	.045	26,852	1,200	.061	7,750	473	20.08	122	2,439	4,122	157.2	19,509	114.6
1931	.040	30,751	1,227	.054	12,216	655	18.02	144	2,541	4,423	168.6	18,665	109.6

[1] Common excavation plus other items expressed as equivalent common excavation. (See p. 85 and table 8.)
[2] Portland cement concrete plus other items expressed as equivalent portland cement concrete. (See p. 85 and table 8.)
[3] Indexes and totals were calculated with the bid prices carried to one more decimal place than that to which they are shown in this table.

The effect of these and of other influences which have affected highway design is shown in the index of highway usage which is given in graphical form in figure 2 and in statistical form in table 13.

THE INDEX OF HIGHWAY COST

The cost at which highways are built integrates changes in price and changes in usage. In the highway field the number of units handled in building a mile of finished pavement has been increasing steadily. The price at which these units have been handled has been falling. Cost has remained relatively uniform. The cost index is obtained by applying the annual average quantities handled to the average annual price at which they were handled. In graphical form this index is shown in figure 2. In tabular form it is shown in table 14.

(Continued from p. 80)

the 3-, 6- and 9-ply mats. This conclusion is verified by a consideration of the relative moisture losses sustained, as shown in tables 2 and 4. It has been shown that, for the mats which were wet when applied, the amount of moisture is virtually the same, regardless of the thickness of the mat.

TEST RESULTS SHOW EFFICIENCY OF MATS IN HEAT INSULATION AND CURING

The conclusions drawn from this investigation are necessarily qualified by the fact that the tests were made with small laboratory samples, rather than in the field. The indications are as follows:

1. Cotton mats were effective for heat insulation. Under the conditions of these tests, the 24-hour temperature range under the cotton mats was approximately 35 percent of that which occurred in uncovered concrete and about 60 percent of that which occurred in concrete covered with wet burlap.

2. The insulating properties of the various thicknesses of mat (ranging from 1-ply, about one-third inch thick, to 9-ply, about 3 inches thick) against heat absorbed from the rays of the sun, appeared to be about the same.

3. From the standpoint of strength the cotton mats which were wet once and received no further wetting were as effective in curing as was double-thickness burlap kept wet continuously, the curing period in both cases being 3 days.

4. The cotton mats applied dry were not as effective for curing as were the wet mats or the wet burlap. Specimens cured with dry mats developed about 88 percent of the strength developed by specimens cured under wet mats or under wet burlap kept continously wet.

5. The various thicknesses of mat appeared to be equally effective as curing agents for concrete.

TABLE 4.—*Percentage of original water content of specimens at ages indicated (1-, 2-, and 3-ply mats)* [1]

Curing method	Specimen no.	Percentage of original water content at age of—				
		3 days	10 days	20 days	26 days	28 days
Burlap, kept wet 3 days	1	101.9	90.2	87.9	87.2	97.1
	2	99.8	89.5	87.4	86.4	95.4
	3	101.1	90.1	88.3	87.2	95.9
Average		100.9	89.9	87.9	86.9	96.1
1-ply mat, wet when applied	1	101.1	90.8	87.7	87.2	97.6
	2	97.3	89.1	86.9	85.9	96.5
	3	98.9	90.4	88.7	87.6	97.4
Average		99.1	90.1	87.8	86.9	97.2
2-ply mat, wet when applied	1	101.7	90.3	87.4	86.6	97.5
	2	99.8	89.4	87.5	86.2	96.5
	3	101.6	90.8	89.1	87.7	97.3
Average		101.0	90.2	88.0	86.8	97.1
3-ply mat, wet when applied	1	100.4	88.7	86.3	85.2	95.8
	2	101.7	91.2	88.8	87.5	98.1
	3	100.7	90.3	88.4	87.0	96.7
Average		100.9	90.1	87.8	86.6	96.9

[1] Curing agents removed at end of 3 days, and specimens cured from 3 to 26 days in laboratory air. Specimens immersed in water at end of 26 days.

TABLE 5.—*Results of tests for flexural strength at the age of 28 days (1-, 2-, and 3-ply mats)*

Curing method	Modulus of rupture in pounds per square inch				Strength ratio [1]
	1	2	3	Average	
Burlap, kept wet 3 days	958	1,009	902	956	1.00
1-ply mat, wet when applied	934	896	931	920	.96
2-ply mat, wet when applied	853	962	962	926	.97
3-ply mat, wet when applied	918	880	954	917	.96

[1] Strength developed by double burlap kept wet 3 days taken as standard.

CORRECTIONS

Vol. 14, No. 2, April 1933.—In the article entitled, "The Wisconsin Financial Survey", page 31, in right-hand middle panel of figure: "Vehicle-miles in millions" should read, "Vehicle-miles in billions."

Vol. 14, No. 3, May 1933.—In the article entitled, "The Illinois Financial Survey", page 47, in upper left-hand panel of figure: "Other imposts" should read "all imposts."

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

JUNE 30,1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$					$	$				$	
Alabama	2,344.3	5,032,976.50	2,516,488.13	75	131.2	110.3	241.5						3,199,828.75	Alabama
Arizona	1,270.4	2,884,355.53	1,355,008.46	72	68.0	126.7	194.7	43,050.08	27,982.56		5.9	5.9	1,610.68	Arizona
Arkansas	1,932.8	4,613,104.71	2,192,025.82	72	130.6	89.0	219.6	1,187,545.23	593,772.55	102.1	15.7	117.8	663,644.66	Arkansas
California	2,500.1	7,629,209.24	1,565,804.03	81	130.4	55.1	185.5	73,760.41	17,153.65	.9	2.0	2.9	117,095.88	California
Colorado	1,863.7	2,900,175.77	1,320,286.76	79	116.6	19.1	135.7	394,215.58	177,396.98	27.9	.2	28.1	269,439.58	Colorado
Connecticut	296.8	4,110,880.78	1,695,978.21	80	43.7	5.3	49.0						183,167.11	Connecticut
Delaware	381.1	755,181.00	150,125.44	93	23.6	15.9	39.5							Delaware
Florida	661.1	6,609,972.32	3,168,669.14	81	172.4		172.4						989,164.32	Florida
Georgia	3,238.8	4,303,700.67	1,748,672.41	87	107.4	180.6	288.0						100,636.93	Georgia
Idaho	1,592.4	2,354,542.60	841,030.70	79	90.6	113.9	204.5	81,908.25	41,788.31	6.2	3.4	9.6	73,920.51	Idaho
Illinois	3,110.9	20,162,768.27	7,703,139.50	76	617.0	60.6	677.6	716,775.42	311,902.26	27.0		27.0	95,967.25	Illinois
Indiana	2,109.4	7,139,311.52	2,773,730.55	88	228.3	20.7	249.0	1,372,003.25	133,528.13	65.0	5.3	70.3	209,145.99	Indiana
Iowa	3,540.0	5,321,273.39	803,331.76	96	261.1	38.5	299.6						59,251.82	Iowa
Kansas	4,052.9	3,498,351.61	1,032,386.15	70	190.3	61.0	251.3	163,196.62	75,935.47	8.6	3.1	11.7	153,217.33	Kansas
Kentucky	1,933.6	4,024,383.53	1,395,480.17	73	159.6	122.0	281.6	283,561.74	126,218.05	10.7	13.9	24.6	48,791.33	Kentucky
Louisiana	1,619.6	6,392,245.96	2,785,921.90	68	42.3	24.8	67.1							Louisiana
Maine	823.0	2,203,423.77	508,213.51	74	66.4	.3	66.7	38,958.62	3,989.09		1.9	1.9	76,304.51	Maine
Maryland	872.7	965,442.17	99,880.77	57	33.3	1.5	34.8	251,758.25	25,648.82	5.4		5.4		Maryland
Massachusetts	875.9	4,115,901.96	972,218.48	74	57.2	4.9	62.1						351,632.06	Massachusetts
Michigan	2,339.2	6,674,228.20	2,567,439.95	74	283.9	97.1	381.0	602,750.00	196,440.00	39.3	.8	40.1	175,723.01	Michigan
Minnesota	4,309.6	4,768,398.82	202,728.91	96	150.9	139.9	290.8						34,304.16	Minnesota
Mississippi	1,863.8	6,193,724.28	3,072,313.77	66	185.3	97.9	283.2	250,653.30	125,326.62	11.6	4.2	15.8	2,952,041.86	Mississippi
Missouri	3,233.2	4,351,270.58	737,807.32	47	173.8	17.0	190.8	445,829.85	91,623.60	13.1	16.6	29.7	8,827.28	Missouri
Montana	2,973.2	5,616,576.61	3,142,856.07	76	405.9	255.8	661.7	57,325.91	32,350.96		12.0	12.0	401,817.97	Montana
Nebraska	4,259.7	6,234,000.11	2,913,882.72	88	156.3	159.9	316.2						49,318.84	Nebraska
Nevada	1,352.1	1,975,857.64	721,167.78	91	38.7	127.5	166.2						117,662.21	Nevada
New Hampshire	448.0	563,304.16	232,591.90	75	12.5	.5	13.0	85,798.47	36,415.44	1.6	.7	2.3	64,377.21	New Hampshire
New Jersey	637.9	5,309,569.73	1,735,318.97	87	55.7		55.7						109,558.29	New Jersey
New Mexico	2,300.9	2,532,793.19	975,113.25	96	127.9	103.5	231.4	111,736.03	44,694.41		5.1	5.1	85,464.22	New Mexico
New York	3,516.6	18,271,330.05	5,611,850.00	55	479.1	32.0	511.1						355,781.37	New York
North Carolina	2,358.8	5,040,679.02	2,518,674.96	75	513.8	28.1	541.9	324,785.47	162,392.71	43.4	.3	43.7	977,044.05	North Carolina
North Dakota	5,432.1	3,495,612.32	1,451,921.71	51	237.3	410.3	647.6	299,625.98	123,677.17	22.8	73.5	96.3	416,198.04	North Dakota
Ohio	3,057.9	8,085,323.09	2,179,738.01	78	177.5	52.3	229.8	76,364.00	23,361.08	1.3	3.3	4.6	190,835.32	Ohio
Oklahoma	2,502.2	3,951,579.57	1,210,698.90	84	191.3	74.7	266.0	528,539.65	105,707.88	38.2		38.2	433,937.87	Oklahoma
Oregon	1,630.5	4,104,798.63	1,640,267.03	77	106.6	74.4	181.0	59,172.74	26,004.62	1.7	1.6	3.3	56,880.55	Oregon
Pennsylvania	3,276.8	10,077,698.86	2,532,150.15	68	348.7	14.0	362.7						53,641.77	Pennsylvania
Rhode Island	271.7	1,013,905.12	318,007.01	72	22.5	4.5	27.0						70,834.03	Rhode Island
South Carolina	1,962.5	3,413,612.84	1,277,642.73	85	161.2	154.7	315.9						12,443.92	South Carolina
South Dakota	4,305.3	3,155,035.66	1,288,695.86	78	249.4	201.7	451.1	278,850.45	120,075.71	20.3	29.1	49.4	169,101.27	South Dakota
Tennessee	1,723.5	4,645,392.30	2,321,975.19	72	159.6	40.8	200.4						576,790.79	Tennessee
Texas	8,113.8	15,549,620.20	5,086,325.25	76	551.2	502.8	1,054.0	719,256.12	250,336.03	40.8	18.3	59.1	49,990.22	Texas
Utah	1,286.3	1,877,602.57	774,640.08	84	127.8	36.3	164.1	259,868.69	111,261.16	24.7	12.4	37.1		Utah
Vermont	393.8	366,965.01	43,473.80	83	18.6		18.6	201,364.63	43,000.00	7.6	6.5	14.1	151.97	Vermont
Virginia	1,994.3	4,026,229.27	1,817,348.27	80	179.6	52.7	232.3	35,537.10	17,768.54	2.3		2.3	194,541.87	Virginia
Washington	1,330.7	3,045,261.29	916,420.63	76	109.6	9.7	119.3						59,732.72	Washington
West Virginia	925.6	3,071,207.15	1,256,848.11	83	113.8	8.1	121.9						49,516.08	West Virginia
Wisconsin	2,760.1	5,384,047.69	966,926.70	78	136.3	101.9	238.2	52,061.50	21,500.00		.9	.9	265,183.36	Wisconsin
Wyoming	2,179.8	2,606,875.23	859,481.92	89	215.7	137.9	353.6						151,122.56	Wyoming
Hawaii	109.4	1,687,095.81	1,138,158.71	62	36.9		36.9						514,689.68	Hawaii
TOTALS	107,868.8	242,106,796.30	86,140,857.55	75	8,397.4	3,986.2	12,383.6	8,996,253.34	3,067,251.80	522.5	236.7	759.2	15,190,331.20	TOTALS
Construction completed		176,248,000	64,944,000											
Balance uncompleted		65,859,000	21,197,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

A22.6

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 6 AUGUST 1933

A FOREST ROAD THROUGH THE SHENANDOAH MOUNTAINS NEAR STAUNTON, VIRGINIA

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - See Page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 6 August 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

COMMENTS ON THE HYDROMETER METHOD OF MECHANICAL ANALYSIS

Reported by R. C. THOREEN, Junior Highway Engineer, Division of Tests, United States Bureau of Public Roads

THE DISCUSSIONS comprising this report are suggested by the subgrade soil investigations of the United States Bureau of Public Roads conducted at its laboratories in Arlington, Va., supplemented by data obtained by the author when engaged as assistant in the cooperative subgrade soil research carried on at the Engineering Experiment Station of the Ohio State University, Columbus, Ohio.

The hydrometer method of mechanical analysis, the procedure for which has been published in PUBLIC ROADS (1)[1] has two distinct advantages when used in subgrade soil investigations: First, an analysis by this method requires a minimum of time and effort; and, second, such an analysis furnishes the data required for the construction of a complete particle-size accumulation curve. This, in turn, discloses the percentage of particles of any desired size, whereas with other methods greater effort is required to obtain information on only the sand, silt, and clay contents.

The hydrometer method consists of two operations—the determination of the percentage of suspended soil from hydrometer readings corrected for the particular test conditions and the determination by means of Stokes's law of the maximum particle size corresponding to a particular percentage of suspended soil.

The relation between the hydrometer reading and the percentage of suspended soil is expressed by the formula:

$$P = \frac{(R + \Delta R)a}{W} \times 100 \qquad (1)$$

where

R = hydrometer reading.
W = weight of soil originally dispersed per liter of suspension.
ΔR = correction to hydrometer reading for variation in temperature from 67° F.
a = correction coefficient for variation in specific gravity from 2.65.
P = percentage of originally dispersed soil remaining in suspension.

The maximum grain size in suspension at any given time after dispersion, according to Stokes's law, is expressed by the formula:

$$d = \sqrt{\frac{30nL}{980\,(G - G_1)\,T}} \qquad (2)$$

where

d = maximum grain diameter in millimeters.
n = coefficient of viscosity of the suspending medium, in poises.
L = average distance in centimeters through which soil particles settle in a given time.
T = period of sedimentation, in minutes.
G = specific gravity of soil particles.
G_1 = specific gravity of the suspending medium.

The values of the nine variables—n, L, T, G, G_1, R, W, ΔR, and a—must be available before the percentage of soil in suspension, P, by means of equation 1, and the corresponding grain size, d, by means of equation 2, can be determined.

Both the specific gravity, G_1, and the coefficient of viscosity, n, of the suspending medium, in this case water, are furnished by the Smithsonian Physical Tables (2).

The hydrometer readings, R, corresponding to different times of sedimentation, T, are observed as part of each test. This is true also of the weight of soil dispersed, W, and the specific gravity of the soil particles, G.

This leaves the specific-gravity correction coefficient, a, the temperature correction, ΔR, and the assumed

HYDROMETER METHOD GIVES SATISFACTORY RESULTS IF CAREFULLY APPLIED

Readings of a sensitive hydrometer in soil suspensions give results sufficiently accurate for the determination of the grain size distribution in subgrade soils.

Thorough dispersion of the soil sample is desirable. Apparatus highly efficient for this purpose is now in use.

A high degree of deflocculation is also of importance. Sodium silicate has been found the most satisfactory deflocculating agent.

In the use of Stokes's formula to determine grain size distribution, it is necessary to obtain by experimental methods approximate values for L, the average depth to which soil particles settle in a given time. Satisfactory results were obtained with a value of L equal to 0.42 of the total immersed depth of the Bouyoucos hydrometer, and with a value equal to the depth of the center of volume of the specific gravity hydrometer.

With these values of L, test results were found to be in substantial agreement with the more elaborate pipette method of making the mechanical analysis.

The errors involved in the use of a sensitive hydrometer are subordinate to those due either to partial deflocculation or to the assumptions involved in the use of Stokes's law.

The errors of the hydrometer method are those common to all methods utilizing sedimentation.

[1] Figures in parentheses refer to reports listed in the bibliography attached to this report.

distance of particle settlement, L, to be determined by research before formulas 1 and 2 can be used in a complete procedure for making the mechanical analysis of soils by the hydrometer method. Accordingly, investigations for determining these values were begun early in 1928.

These investigations resulted in the development of a test procedure equally applicable to the Bouyoucos hydrometer and to any sensitive hydrometer calibrated in specific gravities, and provided correction coefficients whereby both the grams per liter concentration and the maximum particle diameter corresponding to the hydrometer readings could be determined. In addition, they demonstrated the fact that the results furnished by the hydrometer method check within practical limits those obtained by the pipette method of the United States Bureau of Chemistry and Soils, and other methods. As a further outgrowth of this work, a new and effective deflocculating agent was introduced and both a new type of hydrometer and a revised dispersing cup and stirring paddle were developed.

The test procedure, as noted above, has been published. Comments on the test procedure and the hydrometer calibration, comparisons of results furnished by different methods of mechanical analysis, discussion of the influence of various deflocculating agents, and descriptions of the new hydrometer and dispersing cup and their use, comprise this report.

THE DETERMINATION OF TEST-PROCEDURE REQUIREMENTS

Interested testing engineers may be curious regarding certain of the requirements of the test procedure as published. The purpose of this report is to present additional explanations of these requirements in order to prevent the duplication of the work on which they are based.

The following requirements are among the most important of those specified in the procedure:

1. Soils having a plasticity index between 0 and 5 are dispersed only five minutes in the dispersing apparatus.
2. Soils having a plasticity index between 5 and 20 are pretreated by soaking in water for 18 hours and then dispersed for 10 minutes.
3. Soils having a plasticity index greater than 20 are pretreated with 6 percent hydrogen peroxide, soaked for 18 hours, and then dispersed for 15 minutes.
4. A sodium-silicate solution is used as a deflocculating agent for all soils.
5. The hydrometer is read at the top of the meniscus formed around its stem.
6. The suspensions are maintained at a constant temperature throughout the test.
7. Readings of the hydrometer are corrected for a suspension temperature other than the standard temperature of the hydrometer.
8. Readings of the hydrometer are corrected for any variation in the specific gravity of the soil from the average or standard specific gravity, 2.65.
9. The depth of settlement, L (equation 2), for the Bouyoucos hydrometer is 0.42 of the distance from the surface of the suspension to the lower end of the hydrometer. The logical and experimental bases of these requirements are discussed in the following paragraphs.

Dispersion.—The results produced by any method of soil analysis based on sedimentation through water depend upon the degree to which the soil particles are dispersed. In the absence of complete dispersion, the rate of settlement will depend upon the sizes of the undispersed aggregates rather than the sizes of the individual particles; and consequently the diameters of accumulations rather than the diameters of individual grains will be indicated.

After the separation of the particles, it is equally important to prevent their flocculating or coalescing into loosely bound aggregates, which settle more rapidly than the individual particles and also lead to error in the size determination.

Soil concretions are more likely to form in the fine-grained than in the coarser-grained soils. Also, the concretions in some fine-grained varieties are likely to offer greater resistance to breaking down than the concretions in other varieties of the fine-grained soils. The colloidal properties which furnish the resistance of aggregations to dispersion are reflected more definitely by the plasticity of the soil than by the fineness of the soil grains.

Thus, a soil with a plasticity index of 8 and a clay content of 77 percent[2] may disperse much more readily than a soil with a plasticity index of 32 and a clay content of but 23 percent.[3]

The higher the plasticity index the greater is likely to be the effort required to disperse the soil. Consequently, it is only natural to disperse soils with high plasticity indices for a longer period than those having little or no plasticity.

The highly efficient dispersing apparatus referred to in the article on test procedure (1) has been found capable of producing the required degree of dispersion of coarse-grained soils (plasticity index, 0 to 5) in 5 minutes. A longer period of dispersion tends to break down the sand grains into smaller fragments and therefore should be avoided.

Moderately plastic soils (plasticity index, 5 to 20) are pretreated by soaking in water for 18 hours before dispersing. The slaking and the softening of the soil aggregations due to this treatment make it possible to disperse these soils properly by agitating for 10 minutes.

The highly plastic soils, with plasticity indices exceeding 20, are likely to contain aggregations very difficult to break down. These soils are treated with hydrogen peroxide prior to the soaking for 18 hours. This pretreatment of the sample separates the aggregated particles to such an extent that a mixing period of 15 minutes is sufficient to produce the desired degree of dispersion. The effect of the hydrogen peroxide pretreatment is shown in table 1.

TABLE 1.—*The effect of hydrogen-peroxide pretreatment on the mechanical analysis of fine-grained soils*

Soil no.	Percentage of sample dispersed			
	No pretreatment; particles finer than—		Hydrogen peroxide pretreatment; particle finer than—	
	0.005 mm	0.002 mm	0.005 mm	0.002 mm
763	34.2	21.5	36.8	26.
791	37.5	18.5	40.5	21.
872	40.0	23.0	44.4	25.
695	46.5	31.0	48.7	35.
837	73.5	62.2	75.4	64.
802	75.5	48.0	78.0	53

[2] See sample no. 6, tables 15 and 16, p. 141, PUBLIC ROADS, vol. 12, no. 5, July 1931.
[3] See sample no. 7, tables 17 and 18, p. 142, PUBLIC ROADS, vol. 12, no. 5, July 1931.

SODIUM SILICATE FOUND BEST DEFLOCCULANT

Deflocculation.—Extensive tests with various deflocculating agents showed sodium silicate to be the most effective for maintaining separation of the dispersed soil particles in suspension. The other reagents tested are those commonly used for deflocculation of soil suspensions and include potassium hydroxide, ammonium hydroxide, sodium carbonate, and sodium oxalate. Results obtained from hydrometer analyses on duplicate samples of four soils treated with the different reagents are shown in table 2. All the samples were dispersed for equal periods, and, as indicated in table 2, the reagents were used in the amounts required to make the suspensions equinormal.[4]

TABLE 2.—*Total percentages of various sizes obtained by treatment with different dispersing agents*

Soil no.	State	No reagent			Potassium hydroxide 5 cc N/1 solution			Sodium carbonate 5 cc N/1 solution			Sodium oxalate 10 cc N/2 solution			Sodium silicate 5 cc N/1 solution		
		Percentage of particles finer than—														
		0.005 mm	0.002 mm	0.001 mm	0.005 mm	0.002 mm	0.001 mm	0.005 mm	0.002 mm	0.001 mm	0.005 mm	0.002 mm	0.001 mm	0.005 mm	0.002 mm	0.001 mm
5792	Iowa	1.5	(1)	(1)	18.5	[2] 5.0	[2] 2.0	20.0	12.0	[2] 7.0	22.0	17.0	15.0	22.0	17.5	15.5
5853	Miss.	50.0	(1)	(1)	66.5	22.5	14.0	82.0	43.0	28.0	78.0	59.5	50.0	87.0	67.0	56.5
6054	N.C.	(1)	(1)	(1)	23.0	[2] 6.0	[2] 3.0	37.0	[2] 8.0	(1)	29.0	[2] 7.0	[2] 3.0	38.0	27.0	22.5
6278	Texas	[2] 9.0	6.0	(1)	19.0	[2] 1.0	(1)	[2] 40.0	[1] 4.0	[1] 1.0	64.0	34.0	(1)	62.0	45.0	29.0

[1] Completely flocculated.
[2] Partially flocculated.

In this case the degree of normality was equal to that furnished by 5 cc of a 1-normal (N/1) solution in a liter of suspension. Due to its low solubility, the sodium oxalate had to be used in a half-normal (N/2) solution. This required 10 cc of the sodium oxalate half-normal solution per liter to make the suspension equinormal with the suspensions containing 5 cc of the 1-normal potassium-hydroxide, sodium-carbonate, and sodium-silicate solutions.

The sodium-silicate solution specified in the procedure is prepared by dissolving chemically pure crystals in sufficient water to give the desired density of 3° Baumé. In the absence of a Baumé hydrometer, the density may be conveniently measured with a Bouyoucos hydrometer, which at 67° F. should read about 36.5 gram divisions at the proper dilution, or with a specific gravity hydrometer, which, if calibrated at 67° F., should read about 1.023.

The properties of the sodium-silicate crystals used are as follows (3): Grayish white in color; formula as identified by the melting point of 48° C., $NaSiO_3 \cdot 9\ H_2O$ (sodium metasilicate); loses 6 H_2O at 100° C.

Reading the hydrometer.—Because of the turbidity of the suspension, the hydrometer is read at the top of the meniscus which rises on the stem. This differs from the usual practice of reading the hydrometer through the liquid, at the point where the surface cuts the hydrometer scale. For this reason each reading would be too small by an amount equal to the height of the meniscus were the suspending medium pure distilled water. Such being the case, a correction equal to the height of the meniscus or approximately 1 division on the hydrometer scale (1 gram per liter) would have to be added to each reading.

In our case, however, the density of the suspending medium is increased by the silicate solution a constant amount equal to a specific gravity of 0.0005 or approximately 0.8 gram per liter. This correction is subtractive and balances for all practical purposes the additive correction of approximately 1 gram per liter referred to above.

The top of the meniscus may be accurately discerned by placing the source of illumination so as to cause a reflection or high light at that point.

It is essential that the hydrometer be kept clean while in use, as grease or soil on the stem will prevent the proper formation of a meniscus, thus making accurate readings difficult to obtain, and causing lag of the instrument. Also, soil accumulating on the bulb will cause erroneous readings.

Constant-temperature bath.—Since the velocity with which a grain of given diameter will settle is a function of the viscosity of the suspending medium and this in turn is dependent on the temperature, it is evident that if n, the coefficient of viscosity (equation 2), is kept a constant throughout the test a more accurate solution of this formula is possible than will be the case if n is a variable. Changes in temperature in the soil suspension also cause convection currents which interfere with the free fall of the particles. Finally, hydrometers should be used at temperatures as close as possible to that at which they were calibrated, in order to avoid errors due to thermal expansion or contraction of the hydrometer bulb.

In view of these facts the soil suspension should be maintained at a uniform temperature which is as close to the standard temperature as is possible without undue refinement of apparatus.

The temperature correction.—The necessary correction of any hydrometer reading for a variation from the standard temperature is determined by the accompanying change in the density of the water. For a specific gravity hydrometer the correction is equal to the difference between the density of water at the standard temperature and the density at the temperature of the suspension under test.

For the Bouyoucos hydrometer, the temperature correction, ΔR (equation 1), is determined by dividing the difference between the density of water at the test temperature and at 67° F. by the density change equivalent to one division on the hydrometer scale.

The density of the suspension when a properly calibrated Bouyoucos hydrometer reads 0.0 is 0.9984, the density of water at 67° F. The density of the suspension for a hydrometer reading of 50 grams per liter equals 1.02956.

Thus the total change in density for the 50 divisions on the hydrometer scale is $1.02956 - 0.99840 = 0.03116$; and the difference for each gram per liter division on the hydrometer scale equal 1/50 of 0.03116 or 0.000623.

The derivation of the corrections for temperature at 10° intervals between 40° F. and 100° F. are given in table 3, and the correction curve (fig. 1) is based on the values in the last column of this table.

The corrections obtained from this curve apply to a hydrometer which reads 0.0 (at the water surface, not at the top of the meniscus) in distilled water at 67° F., the nominal standard temperature for the Bouyoucos

[4] A normal solution (written N/1 or 1 N) is one which contains, in a liter of solution, a number of grams of the dissolved substance equal to the molecular weight divided by the number of replaceable hydrogen atoms or their equivalent. Thus, a normal solution of potassium hydroxide, KOH, molecular weight 56, contains 56 grams per liter of solution, since it has one hydrogen equivalent, K. A solution of sodium carbonate, Na_2CO_3, molecular weight 106, contains 53 grams per liter of solution, a number equal to one half the molecular weight, since it has two hydrogen equivalents, Na_2.

hydrometer. Actually, the reading at 67° F. is likely to vary among different hydrometers [5] in which event this reading, if greater than 0.0, must be subtracted from each of the temperature corrections; if less than 0.0, it must be added.

The reading to be added or subtracted is the reading of the Bouyoucos hydrometer in distilled water at 67° F. The correction curve is then plotted a corresponding number of divisions to the right of 0 correction when the reading is positive and to the left of this location when the reading is negative.

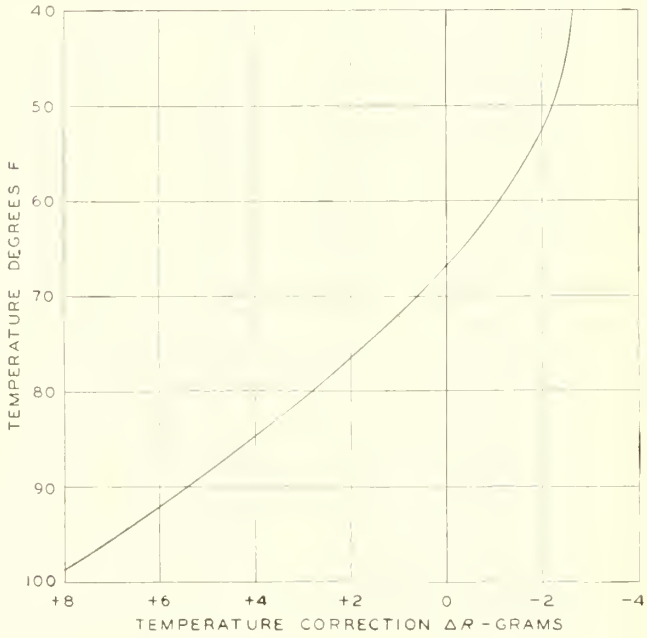

FIGURE 1.—TEMPERATURE CORRECTION CURVE FOR THE BOUYOUCOS HYDROMETER READING.

As an alternate method, the temperature correction curve may be determined by reading the hydrometer in distilled water at successive temperatures between 40° F. and 100° F.

TABLE 3.—*Derivation of temperature corrections for the Bouyoucos hydrometer readings*

Temperature	Density of water	Difference from density at 67° F	Equivalent number of divisions on the hydrometer scale, Diff. / 0.000623	Temperature correction
°F.				
40	0.9999986	0.0016428	2.63	−2.6
50	.9997282	.0013724	2.20	−2.2
60	.9993343	.0006785	1.09	−1.1
67	.9983558	.0000000	0.00	0.0
70	.9980016	.0003542	0.56	+0.6
80	.9966273	.0017285	2.76	+2.8
90	.9949920	.0033638	5.38	+5.4
100	.9930691	.0052867	8.46	+8.5

Calibration of the hydrometer scale.—This operation may be accomplished, if desired, by comparing the readings of the Bouyoucos hydrometer with those of standard density hydrometers, or with the densities of standard solutions. For this purpose the relation between the grams of soil per liter and the corresponding density of the suspension must be determined.

[5] Among seven hydrometers used by the writer in the cooperative subgrade soils laboratory at Ohio State University, this reading varied from 0.0 to −1.5.

The density of a soil suspension equals, in c.g.s. units,

$$\frac{\text{weight of water} + \text{weight of suspended soil}}{\text{volume of water} + \text{volume of suspended soil}}$$

This may be formulated as follows:

$$D = \frac{G_1\left(V - \frac{w}{G}\right) + w}{V} \qquad (3)$$

In which D = density of suspension.

V = volume of suspension in cubic centimeters.
w = weight of suspended soil, grams.
G_1 = density of water.
G = specific gravity of soil.

Substituting 1,000 for V, 2.65 for G, and 0.9984 for G_1, we have,

$$D = \frac{0.9984\left(1{,}000 - \frac{w}{2.65}\right) + w}{1{,}000}$$

$$= \frac{w}{1604.5} + 0.9984 \qquad (4)$$

$$\text{and } w = 1604.5\ (D - .9984) \qquad (5)$$

Equation 5 expresses the basic relation between the grams of soil per liter of suspension and the density for standard test conditions.

Since the Bouyoucos hydrometer is proposed to show the grams per liter under these conditions, equation 5 should express also the relation between the hydrometer readings and the corresponding densities of the suspensions.

The calibration curve of the hydrometer scale, therefore, is the curve which expresses the relation of the observed readings to the values of w computed according to equation 5.

On the basis of the relation between density of suspension and weight of suspended soil, expressed in equation 5, the percentage of originally dispersed soil per liter corresponding to a given density of suspension may be determined from equation 1 by substituting for the quantity $R + \Delta R$ (which equals w for a specific gravity of 2.65) the equivalent value of w in terms of density, 1604.5 $(D + \Delta D) - 0.9984$, in which ΔD equals the correction of the density for a temperature other than 67° F. When this substitution is made, equation 1 becomes

$$P = \frac{1604.5[(D + \Delta D) - 0.9984]a}{W} \times 100 \qquad (6)$$

For example, for the values

$D + \Delta D = 1.0248$, $a = 1.05$, and $W = 50$

$$P = \frac{1604.5(1.0248 - 0.9984)1.05}{50} \times 100 = 88.95 \text{ percent}$$

Specific gravity correction.—Since for a given weight of soil dispersed, the density of the suspension varies with the specific gravity of the soil, the readings of the Bouyoucos hydrometer must also be corrected for a soil gravity other than 2.65 in order to determine the

true grams per liter concentration. The necessity for this correction was first noted in the cooperative subgrade soils laboratory at Ohio State University when the writer attempted to make hydrometer tests on three paint pigments with specific gravities ranging from 4 to 5.5. As a result of the high specific gravities, the 1-minute readings in suspensions of 50 grams per liter ranged from 57 to 62.5.

Since the hydrometer reading for any given weight of suspended soil is determined by the difference between the density of water and that of the soil suspension, readings in a suspension of soil having a gravity other than 2.65 must be corrected in proportion as this difference, or density range, varies from that of a suspension of an equal weight of soil of gravity 2.65.

If the specific gravity, G, of the soil is greater than $G_0=2.65$, the reading of the hydrometer will be such as to give, if uncorrected, a percentage of soil in suspension which exceeds the correct percentage in proportion as the density range exceeds the density range under standard conditions (i.e., when $G=G_0=2.65$). The specific gravity coefficient a is therefore defined by the ratio

$$a=\frac{D_0-G_1}{D-G_1}$$

where

D = density of suspension.

D_0 = density of suspension under standard conditions.

Substituting the expression for density of suspension given in equation 3, we have

$$D-G_1=\frac{G_1\left(V-\frac{w}{G}\right)+w}{V}-G_1$$

$$=\frac{w(G-G_1)}{GV}$$

Similarly, for standard conditions

$$D_0-G_1=\frac{w(G_0-G_1)}{G_0V}$$

The value of a is therefore given by the expression

$$a=\frac{D_0-G_1}{D-G_1}$$

$$=\frac{w(G_0-G_1)}{G_0V}\times\frac{GV}{w(G-G_1)}$$

$$=\frac{G_0-G_1}{G_0}\times\frac{G}{G-G_1}$$

Since $G_0=2.65$ and $G_1=0.9984$,

$$a=\frac{2.6500-0.9984}{2.6500}\times\frac{G}{G-0.9984}.$$

The curve of the values of a for specific gravities between 2.00 and 3.00 is shown in figure 2.

This correction of the hydrometer reading for determining the percentage of soil in suspension was expressed in tabular form in the procedure published in PUBLIC ROADS, vol. 12, no. 8. It should not be confused with curve C, figure 3, in the same publication,

FIGURE 2.—SPECIFIC GRAVITY CORRECTION CURVE FOR HYDROMETER READING.

which shows specific gravity corrections to be used for determining the soil particle size.

The distance of particle settlement, L.—It is well to recall at this point that the hydrometer method of analysis consists of two separate and distinct operations: (a) The determination of the percentage of soil in suspension after any period of sedimentation and (b) the determination of the corresponding maximum grain size.

The preceding discussion has brought out the fact that the determination of the percentage of the soil in suspension is made according to well established physical laws and hence can be considered in the nature of an exact operation.

In contrast, the use of Stokes's law in the determination of particle size, requires so many assumptions that the value of L becomes entirely arbitrary and can be determined only by comparison with other methods of determining grain size.

BASIC ASSUMPTIONS OF STOKES'S LAW DISCUSSED

A brief statement of the assumptions embodied in the use of Stokes's formula and their validity will serve to make clear its limitations, especially when applied to the hydrometer method of soil analysis. These assumptions, according to Searle (4) and Keen (5), are as follows:

1. The particles of soil matter are much larger than the particles of the liquid. This holds true for only those particles whose size is not so minute that they begin to show Brownian movement.

2. The liquid is of infinite extent in comparison with the settling particles, i.e., the motion is not affected by the proximity of the containing walls or of neighboring particles. Usually the vessel is large enough to overcome the effect of the containing walls, but high concentrations of suspended soil may interfere with the free fall of the particles.

3. The particles are smooth and rigid spheres. Soil particles can be considered rigid, but they are not necessarily smooth, and are rarely spherical.

4. No slipping occurs between the particles and the liquid. This assumption can be taken as correct, since soil particles are wetted by water.

5. The settling velocity of the particles is small. This can be considered true, as only the smaller particles (with low settling velocities) are usually considered, those of larger size being graded by means of sieves rather than on the basis of their settling velocities.

6. The particles are small, but not excessively so, since those of colloidal size are influenced by their Brownian movement. Stokes's formula is not applicable to colloidal particles unless corrected by a factor depending on the collisions between particles, a correction which is not usually made.

7. The particles settle as individuals and not as aggregations, such as may be formed due to flocculation. Flocculation may usually be prevented by use of a chemical reagent; an apparently deflocculated suspension, however, may contain some flocculated clay particles not discernible by ordinary means, which will lead to error because the aggregations do not settle at a normal rate, and may absorb and carry with them fine sand and silt particles.

8. The coefficient of viscosity of the suspending medium is assumed as being that of water. As a matter of fact the viscosity may vary from that of water because of soluble material in the soil, unless this is first removed by washing, or because of the presence of colloidal material fine enough to remain suspended indefinitely.

9. A suspension at any given depth, L, after a given time, T, contains: (a) no particles whose velocity of fall exceeds L/T, and (b) in the same concentration as when originally dispersed, all particles with a velocity of L/T or smaller.

Therefore, at a given time, T, the suspension at any depth, L, is assumed as representative of the original concentration of that portion of the soil particles smaller than a particular maximum grain size whose velocity of fall is L/T. This assumption is not strictly true, because in high concentrations the fall of the large particles is likely to increase the fall of the smaller particles by contact; and in the same manner the smaller particles are likely to decrease the rate of fall of the larger particles.

10. There is a plane of maximum grain size at a depth, L, representative of the percentage of soil in suspension revealed by the hydrometer.

Such a plane exists, but its location with respect to the position of the hydrometer varies with the shape of the hydrometer and the grading of the soil. Therefore, any selected value of L can only be considered as indicating the average position of this plane for all soils and this must be determined by trial.

To make the results obtained by the hydrometer method agree with those furnished by any other method, it is only necessary to substitute in equation 2 those values of L which indicate for a given percentage in suspension the same maximum grain sizes as are indicated by the other method of analysis.

VALUE OF L DETERMINED BY COMPARISON WITH PIPETTE METHOD

In the present instance it was desired to determine that value of L which, when substituted in Stokes's formula, indicates for given percentages in suspension the same maximum grain sizes as those indicated by the pipette method of the Bureau of Chemistry and Soils. This was desired, not because the agronomists' classifications are directly applicable for use in subgrade surveys but because of the great amount of available published information on soil surveys made by that Bureau and the very important part the mechanical analysis plays in its soil classification.

In the first investigation at Arlington, L was tentatively assumed to equal the distance from the surface of the suspension to the location of the center of volume of the hydrometer system in the suspension.

Tests in the laboratory of the Bureau of Public Roads disclosed that the clay contents indicated by the use of this value of L agreed fairly well with those indicated by a method formerly used by the Bureau of Chemistry and Soils (6). They were appreciably smaller, however, than the clay contents indicated by the pipette method now used by the Bureau of Chemistry and Soils (7). Consequently, a second trial value of L was investigated.

Since at any time after the beginning of the test the density varies from top to bottom, it seemed logical to assume that the required maximum diameter will be located at the plane where the density is equal to the average density of that portion of the suspension supporting the hydrometer.

Additional investigation in the laboratory of the Bureau of Public Roads, by means of pipette extractions, disclosed that the plane of average density of the portion of the suspension supporting the hydrometer varies from 0.33 for some soils to 0.50 of the distance from the surface of the suspension to the bottom of the hydrometer for other soils. Therefore a value of L equal to 0.42 of the immersed depth of the hydrometer may be taken as the average distance from the surface of the suspension to the plane of average density of that portion of the suspension supporting the hydrometer. This value was selected as a second trial value of L.

The hydrometer method, with this new value of L, gave results in substantial agreement with those obtained by the Bureau of Chemistry and Soils.

TABLE 4.—*Comparison of the corrected Bouyoucos hydrometer reading with the concentration in grams per liter at the reference point*

Soil no.	Time	Hydrometer reading	Grams per liter at 0.42 of distance from surface to bottom of hydrometer
	Hours		
4056	1	28.0	27.(
	4	22.0	23.(
	24	10.5	11.(
4281	1	43.0	42.(
	4	32.5	33.5
	24	17.0	15.(
602	1	14.2	12.5
	4	9.3	7.5
	24	4.0	3.5
683	1	13.5	12.(
	4	9.1	8.(
	24	6.2	5.(
720	1	20.4	20.5
	4	15.4	15.(
	24	11.7	10.5
770	1	23.0	21.5
	4	17.9	16.(
	24	11.2	10.(
808	1	21.9	21.(
	4	17.6	17.(
	24	12.8	10.(
842	1	15.8	15.(
	4	13.5	12.(
	24	10.0	9.(
965	1	25.7	26.(
	4	19.3	19.(
	24	10.8	11.(

As illustrated in table 4, the agreement between the corrected hydrometer reading and the concentration in grams per liter indicated by pipette extractions was found to substantiate the assumption that the plane of the average density of that portion of the suspension holding the hydrometer is located a distance from the top of the suspension equal to the selected value of L.

Subsequently a third investigation of the required value of L was made. In this case the hydrometer reading R, required to give the percentage of a particular fraction (e.g., clay, 0.005 mm) obtained by the pipette method was determined from equation 1 by substituting the percentage, P, of the fraction, together with the necessary temperature and specific gravity corrections, ΔR and a, and weight of soil, W, used in the hydrometer test. The time interval, T, corresponding to this reading was then determined from the curve of original hydrometer readings plotted against time. With this value substituted for T, and the particular grain diameter substituted for d, Stokes's formula was then solved for L, the depth of settlement. The ratio of this depth to the immersed depth of the hydrometer gave L as a fraction of the distance from the surface of the suspension to the bottom of the hydrometer.

A number of determinations made in this manner by use of hydrometer test data in conjunction with results furnished by the Bureau of Chemistry and Soils gave an average value for L equal to 0.419 of the immersed depth of the hydrometer. Since this value agreed with the selected value of L, equal to 0.42 of the immersed depth, the latter, which had now been in use for some time, was adopted for use in the procedure as published.

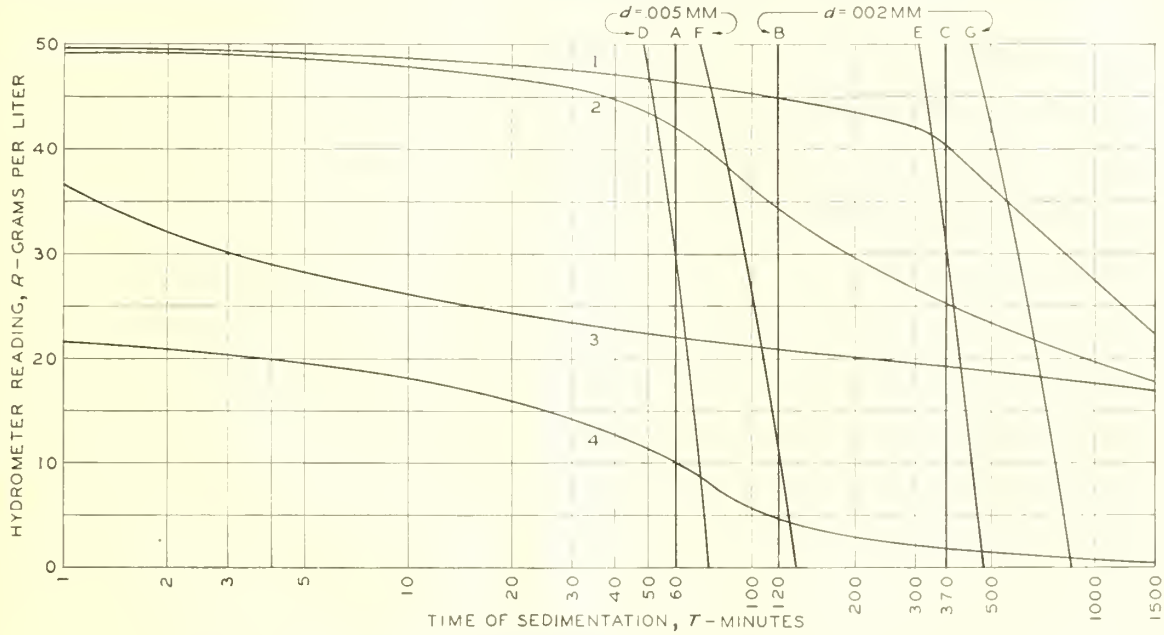

FIGURE 3.—VARIOUS TIMES OF SEDIMENTATION FOR CLAY AND FINE CLAY.

DEPTHS OF SETTLEMENT OF GRAINS OF A PARTICULAR DIAMETER

According to Stokes's law L, the distance of particle settlement, is a function of T, the time of settlement. Therefore, the distance of settlement for grains of any particular diameter may be expressed also in terms of T.

The values of T corresponding to the values of L selected for use in the Bureau of Public Roads procedures (8) vary between 43.5 and 75.6 minutes for the determination of the total clay content (0.005 to 0 mm) and between 272 and 473 minutes for the determination of the fine clay content (0.002 to 0 mm) for hydrometer readings of 60 and 0, respectively.

To accomplish the same results Bouyoucos (9) suggested that a time, T, of 1 hour, be used to determine the total clay content and a time of 2 hours be used to determine the fine clay content. Eno (10) suggested a time, T, of 60 minutes for the total clay content and 370 minutes for the fine clay content.

In addition, times varying from 57.5 to 137.2 minutes (depending on the depth of the hydrometer) for the total clay content, and from 359 to 857 minutes for the fine clay content, were found in the first investigation of the method by the Bureau of Public Roads, in which the reference plane was assumed to be located at the center of volume of the hydrometer system. The depths of settlement, L, of the clay and fine clay particles, corresponding to the various values of T referred to above, as computed for standard test conditions, are shown in table 5.

TABLE 5.—*Distance of Settlement, L, of clay and fine-clay particles for various suggested times, T*

Clay: Particles finer than 0.005 mm		Fine clay: Particles finer than 0.002 mm	
T	L	T	L
Minutes	*Centimeters*	*Minutes*	*Centimeters*
60	7.93	120	2.53
43.5	5.75	370	7.83
75.6	10.00	272	5.75
57.5	7.60	473	10.00
137.2	18.15	359	7.60
-		857	18.15

The significance of the use of the different suggested times of sedimentation for determining the clay and fine-clay fraction is illustrated in figure 3, which shows the relation between hydrometer reading, time of sedimentation as a function of L, and grain diameters of 0.005 mm and 0.002 mm. Line A indicates the time of

sedimentation for clay (0.005 mm) suggested by Bouyoucos and Eno. Line B is the time of sedimentation for fine clay (0.002 mm) suggested by Bouyoucos; and line C the time for fine clay suggested by Eno. These three lines give constant grain diameter for all positions of the hydrometer.

Lines F and G indicate the times of sedimentation for clay and fine clay corresponding to values of L equal to the distance from the surface of the suspension to the center of volume of the hydrometer, as used in the first investigations by the Bureau of Public Roads.

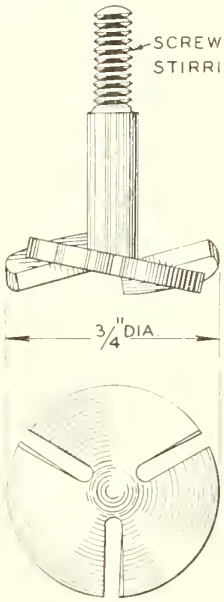

FIGURE 4.—DETAIL OF STIRRING PADDLE.

Lines D and E indicate the times of sedimentation for the clay and fine-clay fractions corresponding to the values of L equal to 0.42 of the distance from the surface of the suspension to the bottom of the hydrometer as specified in the Bureau's procedure.

As compared with curve A (60-minute sedimentation) the times corresponding to the selected L, curve D, figure 3, are such as to indicate higher clay contents for soils with more than about 60 percent of clay; lower clay contents for soils with less than about 60 percent; and the same clay contents for soils with about 60 percent of clay.

For moderately heavy clay soils the clay contents based on the 60-minute reading, line A, figure 3, were in substantial agreement with those indicated by the pipette method. On the same basis, however, the clay contents of very fat clays were indicated to be slightly low and those of very lean soils were indicated to be somewhat high when compared with the results furnished by the pipette method. Therefore, the times (line D) which change with the hydrometer reading indicate clay contents in closer agreement with those indicated by the pipette method than the 60-minute reading (line A).

Curves 1, 2, 3, and 4, figure 3 and table 6, illustrate how the effect of variations in L depends upon the grading of the soil sample. Here it can be seen that L is critical only for those portions of the curves of hydrometer reading against time which slope appreciably. Consequently, if primary interest is in the determination of the clay and the fine clay contents, L is critical for those soils whose curves of hydrometer reading against time slope appreciably in the vicinity of either the 60- or 370-minute ordinates. Thus L is critical for the fine-clay content for the soil curve 1; for both clay and fine-clay content, curve 2; and for the clay content, curve 4.

TABLE 6.—*Percentage of clay (0.005 mm) and fine clay (0.002 mm) as determined by various times of sedimentation. (Fig. 3)*

Soil curve no.	Percentage clay (0.005 mm) according to line—			Percentage fine clay (0.002 mm) according to line—			
	A	D	F	B	C	E	G
1	93.0	93.6	92.0	89.8	81.0	82.8	70.0
2	83.8	86.4	77.0	69.0	50.6	49.8	43.8
3	44.2	44.0	42.0	42.0	38.6	38.0	36.4
4	20.0	17.0	8.6	9.4	3.6	3.0	1.6

NEW APPARATUS DEVELOPED IN CONNECTION WITH THE HYDROMETER INVESTIGATIONS

Improvements in dispersing apparatus.—The stirring paddle furnished with the dispersing apparatus tends to wear out rather rapidly, especially when sandy soils are dispersed. This necessitates frequent changing of the paddle in order that the efficiency of the apparatus shall not be impaired. Furthermore, the paddle

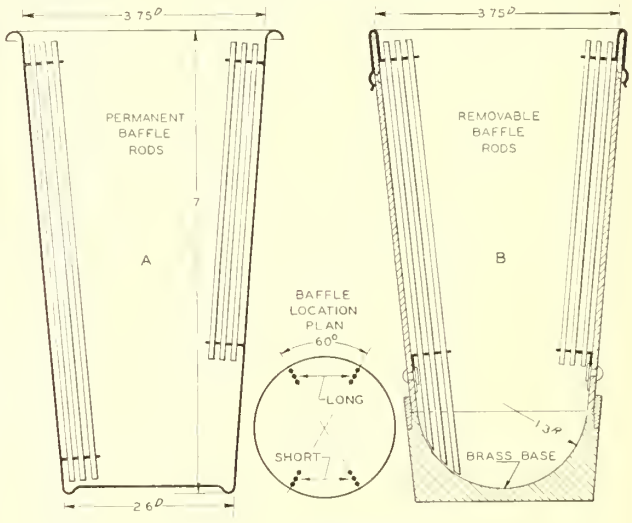

FIGURE 5.—DISPERSION CUPS.

of the dispersing apparatus as purchased is an integral part of the stirring shaft. Therefore, it was necessary to replace the entire stirring rod in order to replace the paddle. A new paddle (fig. 4), made of wear-resisting material and designed to thread into the end of the stirring rod, makes the necessity of replacing paddles less frequent and eliminates the necessity of changing the stirring rod with the paddle.

There was a tendency for some soils to accumulate in the bottom and around the baffles of the original flat-bottom dispersing cup (see fig. 5, left). To overcome this difficulty a new cup with a semispherical bottom and detachable baffles has been recently designed and constructed for use in the Bureau's soils laboratory (see fig. 5, right).

Preliminary tests made with this cup indicate that the shape exerts an influence on the degree of dispersion. In these tests duplicate samples of each of two soils were agitated for equal periods in the two cups and tested with a hydrometer. These periods were purposely

shortened so that the soil would not be completely dispersed and comparative results would be obtained. These results are given in table 7. The higher readings indicate that the concave bottom facilitates dispersion of the soil sample.

TABLE 7.—*Hydrometer readings in suspensions of duplicate samples dispersed (1) in original cup and (2) in cup of new design*

Soil No. 1			Soil No. 2		
Dispersed 5 minutes each			Dispersed 3 minutes each		
Time	Hydrometer reading		Time	Hydrometer reading	
	(1)	(2)		(1)	(2)
5 minutes	40.0	40.9	10 minutes	24.7	25.6
15 minutes	38.5	39.4	1 hour	16.0	18.5
18 hours	26.4	28.0	3 hours	12.0	14.5
42 hours	19.5	21.7	24 hours	6.3	7.8

The new hydrometers.—The new hydrometers for soil analyses developed as an outgrowth of the Bureau's soil investigations are made with a bulb of special shape. These hydrometers are calibrated to read specific gravity directly on the basis of a water density of 1 at 67° F. They are calibrated at this temperature for convenience in comparing test results with those furnished by the Bouyoucos hydrometer. One hydrometer has a range of 0.995 to 1.050 specific gravity, which covers extreme densities of suspensions due to high specific gravities of suspended material or use of large samples. The other two hydrometers cover ranges of 0.995 to 1.020 and 1.015 to 1.040, respectively, and are intended for more accurate analytical work.

The general design of these hydrometers is shown in figure 6, A, which illustrates the first hydrometer mentioned, as compared with the Bouyoucos hydrometer (fig. 6, B). The specific-gravity hydrometers are loaded with mercury confined by a glass septum according to a design patented by the manufacturer. The shape of the bulb is designed to minimize disturbance of the suspension when the hydrometer is inserted or removed.

The percentage of soil in suspension corresponding to a given reading of a specific gravity hydrometer is determined from equation 6, by substituting the hydrometer reading for D. Since the new hydrometers are calibrated on the basis of a water density of 1.0 at 67° F., it is also necessary to substitute this value for the absolute density 0.9984, in the equation. The temperature correction, ΔD, is equal to the difference in the relative density of water at 67° F. and at the temperature of the suspension under test. The curve of temperature corrections for the specific gravity hydrometer is shown in figure 7.

The formula for the percentage of soil in suspension corresponding to a reading of the specific gravity hydrometer is written:

$$P = \frac{1604.5\ (D + \Delta D - 1)a}{W} \times 100 \qquad (7)$$

The value of L for use in the computation of the maximum grain diameter corresponding to a given reading of a specific gravity hydrometer is the distance from the surface of the suspension to the elevation of the absolute center of volume of the hydrometer. This

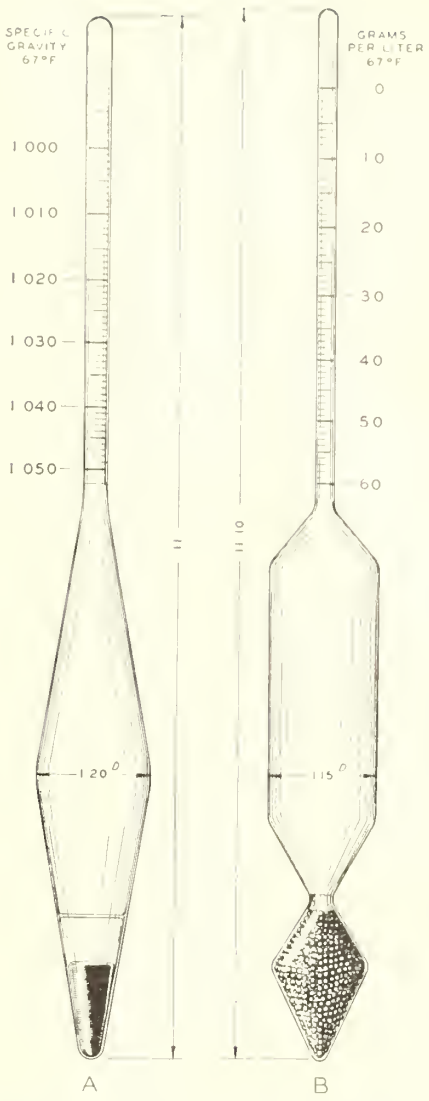

FIGURE 6.—SPECIFIC GRAVITY AND BOUYOUCOS HYDROMETERS COMPARED.

is because the differences between the distances from the surface of the suspension to the absolute center of volume and to the center of volume of the immersed portion of the hydrometer are so small that they do not affect the results appreciably.

Subsequently, it is shown that, probably because of the difference in the shapes of the two hydrometers, results obtained by the use of the specific gravity hydrometer, with L as designated, are in substantial agreement with those furnished by the Bouyoucos hydrometer, with L equal to 0.42 of the immersed depth.

The laboratory test procedure for the specific-gravity hydrometer is the same as that used for the Bouyoucos hydrometer. The formulas given for the determination of the percentages and the grain diameters serve as a basis for the construction of the necessary charts for the graphical solution of the data.

These charts are like those used for solution of the test data obtained with the Bouyoucos hydrometer, except that density values are substituted for grams per liter on the vertical scales, and the curves of time of sedimentation in the chart for the grain diameter determinations are based on the distance to the center of

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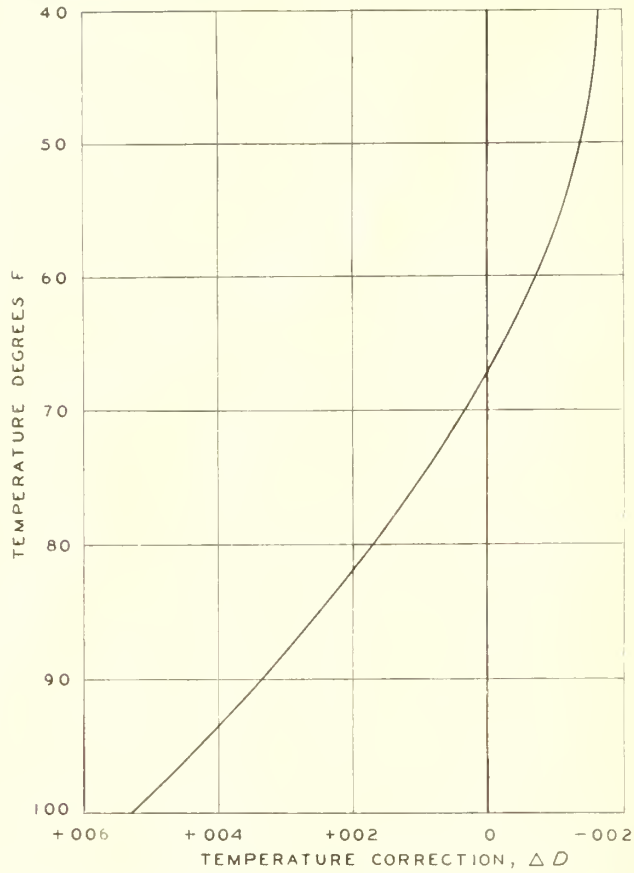

FIGURE 7.—TEMPERATURE CORRECTION CURVE FOR THE SPECIFIC GRAVITY HYDROMETER READING.

volume rather than on 0.42 of the distance to the lower end. With these revised charts the graphical solution may be performed with the same apparatus and in the same manner as in the case of the Bouyoucos hydrometer, as described in the published report on the graphical solution (8).

AGREEMENT OF RESULTS OBTAINED BY DIFFERENT METHODS OF ANALYSIS

As a final step in the mechanical analysis investigation, the results furnished by the hydrometer method were compared with those furnished by three other sedimentation methods.

The sedimentation methods were:

(*a*) The pipette method of the Bureau of Chemistry and Soils.

(*b*) A modified pipette method employed in the laboratory of the Bureau of Public Roads.

(*c*) A beaker method employed at the subgrade soils laboratory of the Ohio State University.

PIPETTE METHOD DESCRIBED

The analyses by the method of the Bureau of Chemistry and Soils were performed in the laboratories of that Bureau. The essential features of this method are as follows: Two samples of that portion of the air-dried soil passing a 2-millimeter screen are weighed out, a 10-gram sample for the analysis and a 5-gram sample for the determintion of hygroscopic moisture.

The 10-gram sample is placed in a 250-cubic centimeter Pyrex electrolytic beaker (diameter 5 cm, height 13 cm) and 6 percent hydrogen peroxide is added in sufficient amount (40 cc or more) to decompose the organic matter. The removal of the organic matter is indicated by a change in color of the soil from dark to light, caused by the removal of the black organic material from the soil. The beaker, containing the sample and peroxide, is warmed in a slow steam bath[6] over night. Then if all organic matter does not appear to have been removed, more hydrogen peroxide of full strength (30 percent) is added. After decomposition of the organic matter, the excess peroxide is boiled off.

The soil sample is then washed as follows: The contents of the beaker are filtered by means of a short Pasteur-Chamberland filter, the residue washed by stirring with 125 cc of distilled water and again filtered, this process being repeated six times. After each filtration the soil adhering to the filter is removed from the filter by applying back pressure, and replaced in the beaker.

Following the last washing, the soil sample is evaporated to dryness and then oven dried at 105° C. for 16 hours (over night). After cooling in a dessicator, the beaker and sample are weighed to the nearest milligram, the difference between the weight and that of the beaker alone being the weight of sample after organic matter and solution loss.

The sample is then transferred to a 250 cc nursing bottle, 10 cc of 0.5 N sodium oxalate solution added as a deflocculating agent, and the volume made up with distilled water to 150 cc. The bottle is then placed in a shaking machine and shaken over night.

When the shaking is completed, the contents are washed through a small sieve fitted with Tyler 300-mesh wire screen cloth (size of openings 0.05 mm, square) into a 1-liter graduated cylinder, which is then filled to the 1-liter mark with distilled water and set aside for sedimentation.

The material remaining on the sieve (sand plus some silt) is dried and weighed and separated by sieves into size classes.

The contents of the sedimentation chamber are stirred with a motor-driven propeller, and by means of a 25-cc automatic pipette a sample is immediately withdrawn before any of the soil particles have had time to settle out. The pipette is emptied into a weighing dish and two or three washings from the pipette added.

The time required for a particle 0.005 mm in diameter to settle 10 cm, is computed from Stokes's formula, using a density of 2.61 and the viscosity of water for various temperatures. From a curve plotted with temperatures as ordinates and time as abscissae, the time for pipetting the clay is determined (77 minutes at 20° C.). At this time the pipette is lowered into the suspension until the tip is 10 cm below the liquid surface, filled by an even suction in about 40 seconds, and the entire contents plus two or three washings are run into a weighing dish.

Pipetting of the fine clay (0.002 mm) is similar to that of the clay except that the settling time is fixed at 6½ hours, the necessary depth being determined from a graph based on Stokes's formula, showing the relation of depth to temperature for this constant time. Weighings of the evaporated and oven-dried pipetted samples are carried to 0.1 milligram.

The results of the analysis are calculated as follows: From the moisture determination of the 5-gram sample, the dry weight of the sample for analysis is computed. The difference between this weight and that after peroxide treatment and washing represents the hydrogen-peroxide solution loss. The first dry weight is

[6] A tank with openings in the top to permit insertion of the beakers used, and having a steam connection for applying a constant low heat.

taken as the base for calculation of results except when the analysis is to be used for textural classification, in which event the dry weight after washing is used.

From the dry weight of the first pipetted sample, after deducting the weight of sodium oxalate in 25 cc of suspension, the dry weight of silt and clay in the sedimentation chamber may be directly determined. This is done by multiplying the dry weight of the pipetted sample by a factor equal to the ratio of the total volume of the suspension to the volume of the pipette. In routine analyses the weight of silt and clay in the cylinder is determined by the difference in the weight of the sample before dispersion and the dry weight of the material retained on the 300-mesh sieve. The weight determined by pipetting is used as a check, the maximum allowable difference being 2 percent.

The total weights of clay and fine clay are determined from the dry weight of the second and third pipetted samples in the same manner as that used in the determinations of clay and silt from the first pipetted sample. If the weight of clay is deducted from the total dry weight in the sedimentation chamber, the difference is silt. This, added to the silt sifted from the sand, gives total silt.

MODIFIED METHOD USED IN BUREAU LABORATORY

The modified pipette method of analysis used in the Bureau of Public Roads laboratory differs from the standard method just described in several respects as follows:

1. A sample of soil weighing 50 grams is used in the Bureau method, whereas but 10 grams of soil are used in the standard method. The volume of the suspension in each case is 1 liter.

2. The sample is freed from organic matter in the standard method, and but partially freed of this material in the Bureau method.

3. The sample is washed free of all water-soluble material in the standard method, but not in the Bureau method.

4. The sample is dispersed by a mechanical shaking of the container in the standard method, and by agitation within the suspension itself in the Bureau method.

5. The deflocculating agent used in the standard method is sodium oxalate; in the Bureau method, sodium silicate.

6. In the Bureau method the entire sample is placed in the sedimentation cylinder, the sand being separated by means of a fine sieve following the sedimentation.

7. In the Bureau method, sampling of the suspension immediately after dispersion is omitted. In addition to the clay and colloid fractions, the percentages finer than 0.040 millimeter and 0.001 millimeter are determined by pipetting in the Bureau method, but not in the standard method.

BEAKER METHOD DESCRIBED

The beaker method of analysis is performed as follows:

At the completion of the hydrometer test the suspension is transferred from the testing cylinder into a beaker of 1,000 cc capacity, and the soil again brought into suspension by stirring with a stiff brush. After a given time, T_1, in this case 52 minutes (at 25° C.), the top 8 cm of the suspension is decanted carefully into a large bottle. The decanted liquid contains no particles with diameters greater than d_1, the diameter of the particle which, according to Stokes's Law, settles 8 cm in the suspension in the time, 52 minutes, noted above. The portion of the suspension remaining in the beaker contains all particles of greater diameter than the calculated one (0.005 mm in this case) together with those of lesser size whose settling velocities and distance of settlement were sufficient to carry them below the upper 8 cm of the suspension in the time T_1.

The volume of the suspension in the beaker is brought to 1,000 cc with distilled water, the suspension is again stirred with a brush and after 52 minutes the top 8 cm are again decanted into the bottle mentioned above.

This process is continued until the decanted water is clear after 52 minutes' sedimentation, indicating that all material finer than 0.005 mm (clay) has been washed out. The total amount of this fraction is determined by evaporating a representative sample of the washwater to dryness.

The washing and decantation are continued, with successive sedimentation periods of 26.5, 13.0, and 3.25 minutes to remove, respectively, the fractions of size 0.005 to 0.007 mm, 0.007 to 0.010 mm, and 0.010 to 0.020 mm, the amount of each of these fractions being determined in the same manner as in the case of the clay.

The material coarser than 0.020 mm (sediment from the last washing) is dried and separated into two fractions by a no. 200 sieve. In the present instance the total of the determined fractions checked the original weight of the sample within one percent.

As has been shown, the pipette, beaker, and hydrometer methods are all based on Stokes's formula as applied to sedimentation of particles through water. Nevertheless, these sedimentation methods differ widely from the hydrometer method in the application of Stokes's law. Differences in test procedures prevent an exact agreement of results given by different sedimentation methods.

In the sedimentation methods the values of L are definite, being equal to the distance from the surface of the suspension to the level from which the sample is pipetted, or to which the suspension is decanted. In the hydrometer method, in contrast, the values of L are not definite but must be assumed on the basis of some relation to the elevation of the hydrometer in the suspension.

Furthermore, it has been shown that differences in the treatment of the sample and the method of dispersion may cause differences in results. Thus, for instance, the clay contents indicated by the procedure of the Bureau of Chemistry and Soils and by the beaker method were found to be slightly higher than those furnished by pipetting samples of the suspension used in the hydrometer test.

In the hydrometer method of analysis, therefore, no one value of L can be considered exact for all soils, but instead a compromise among several values indicated desirable by various investigations must be employed. Furthermore, the values of L suitable for a hydrometer of one shape may not be suitable for hydrometers of a different shape. Finally, in the case of uniformly graded soils, the value of L may change appreciably without affecting materially the practical value of the results.

It has been found that an L for the specific gravity hydrometer equal to the distance from the surface of the suspension to the center of volume of the hydrometer and one equal ot 0.60 of this distance for the Bouyoucos hydrometer may serve to give results in substantial agreement with those given by the selected value of L previously discussed (p. 98). This is illustrated in table 8. The different values of L used in table 8 are shown in table 9.

TABLE 8.—*Clay and fine-clay contents determined by use of various values of L in Bouyoucos and specific gravity hydrometer tests*

Soil no.	Below 0.005 mm				Below 0.002 mm			
	Bouyoucos hydrometer		Specific gravity hydrometer		Bouyoucos hydrometer		Specific gravity hydrometer	
	0.42× HD [1]	0.60× CV [2]	0.62× HD	1.00× CV	0.42× HD	0.60× CV	0.62× HD	1.00× CV
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	12	12	13	13	8	8	10	10
2	28	28	28	28	17	17	17	17
3	32	32	34	34	26	26	27	27
4	40	40	41	39	29	29	30	30
5	44	44	44	44	33	33	33	33
6	57	57	54	53	37	36	36	36
7	80	81	77	77	62	63	60	60
8	86	87	87	87	64	64	61	61

[1] Total immersed depth.
[2] Depth of center of volume.

TABLE 9.—*Depths corresponding to different values of L for various hydrometer readings*

Bouyoucos hydrometer			Specific gravity hydrometer		
Hydrometer reading	L=0.42× immersed depth	L=0.60× depth of center of volume	Hydrometer reading	L=0.62× immersed depth	L=depth of center volume
	Cm	*Cm*		*Cm*	*Cm*
0	10.0	12.3	1.000	15.0	16.4
10	9.2	10.9	1.010	13.9	14.7
20	8.5	9.6	1.020	12.8	13.0
30	7.8	8.5	1.030	11.8	11.3
40	7.1	7.3	1.040	10.8	9.6
50	6.4	6.2	1.050	9.8	7.9

TABLES SHOW AGREEMENT OF HYDROMETER METHOD WITH OTHER METHODS OF MECHANICAL ANALYSIS

The agreement of the results given by the hydrometer method with those given by other methods is shown in tables 10, 11, and 12.

TABLE 10.—*Comparison of analyses by (1) Bouyoucos hydrometer (L=0.42 HD) and (2) pipette method (Bureau of Chemistry and Soils Laboratory)*

Soil no.	Percentage finer than—					
	0.050 mm		0.005 mm		0.002 mm	
	(1)	(2)	(1)	(2)	(1)	(2)
487	57.5	56.7	18.0	16.4	13.8	10.7
630	80.6	83.3	39.8	40.7	27.0	26.1
659	76.5	78.9	22.1	20.8	15.1	14.7
695	82.5	84.7	47.0	47.6	35.5	38.3
763	68.8	66.7	35.6	37.9	25.8	30.3
802	98.6	99.6	78.1	78.1	54.0	57.4
837	94.1	95.0	75.0	71.9	64.1	61.9
872	88.0	88.9	44.8	40.6	26.1	29.3
4258	81.0	84.9	21.2	22.2	13.6	11.3
4295	95.0	96.7	38.0	32.3	29.2	27.8
4907	11.6	12.5	7.5	9.0	6.5	3.9
4913	58.5	55.6	24.2	27.2	14.4	19.3
Average deviation from (2)	−0.9		+0.6		−0.5	
Maximum deviation from (2)	−3.9		+5.7		−4.9	

Table 10 gives the results of the hydrometer tests and of pipette analyses by the method of the Bureau of Chemistry and Soils, together with the average and maximum deviations in percentages of the different sizes.

Here the comparison of the hydrometer test results with those of the particular method noted shows agreement within 1 or 2 percent in the majority of cases. Only in very exceptional cases does the variation exceed 5 percent. This agreement with the pipette method is entirely satisfactory for all practical purposes.

Tables 11 and 12 show that the hydrometer results are approximately 3 percent higher on the average than those given by the modified pipette method and between 1 and 2 percent higher than those given by the beaker method.

TABLE 11.—*Comparison of analyses by (1) Bouyoucos hydrometer (L=0.42 HD), (2) Specific Gravity hydrometer (L=1.00 CV) and (3) pipette method (Bureau of Public Roads laboratory)*

Soil no.	Percentage finer than—											
	0.040 mm			0.005 mm			0.002 mm			0.001 mm		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
5782	98.0	98.0	97.2	86.3	86.7	83.5	63.8	61.0	59.0	43.1	38.5	39.
5822	58.0	57.1	57.2	40.2	39.0	35.5	29.2	30.1	27.3	23.7	26.2	21.7
5833	70.8	70.0	67.3	27.6	28.0	24.0	17.1	16.8	14.6	11.4	12.0	9.7
5838	94.5	93.5	91.0	56.8	53.1	36.8	35.7	32.3	27.1	27.1	25.1	23.3
5842	74.1	72.0	70.6	32.0	34.2	30.9	26.0	27.0	25.2	21.2	22.0	18.7
5854	94.0	95.2	93.6	80.1	77.1	76.8	62.0	59.8	56.8	45.2	43.2	41.9
6062	35.2	32.0	29.2	12.5	13.0	9.8	8.0	9.7	6.0	6.5	6.0	3.7
6098	78.1	77.0	75.4	44.0	43.8	40.0	33.0	33.1	29.0	25.3	25.5	22.0
Average deviation from (3)	+2.6	+1.7		+3.6	+3.0		+3.2	+3.1		+2.8	+2.3	
Maximum deviation from (3)	+6.0	+2.8		+6.5	+4.0		+5.2	+4.2		+3.8	+4.5	

TABLE 12.—*Comparison of analyses by (1) Bouyoucos hydrometer (L=0.42 HD) and (2) beaker method*

Soil no.	Percentage finer than—							
	0.020 mm		0.010 mm		0.007 mm		0.005 mm	
	(1)	(2)	(1)	(2)	(1)	(1)	(1)	(2)
558	25.2	25.3	20.0	20.1	17.5	17.1	15.5	16.0
559	50.8	47.5	36.5	34.6	30.8	29.5	25.8	25.4
560	70.5	69.5	56.5	57.9	51.4	50.8	45.2	45.4
561	65.7	64.6	53.0	52.0	46.5	45.0	41.0	40.0
562	48.6	49.4	35.4	34.9	30.0	28.4	25.5	25.8
563	59.0	55.1	46.0	41.6	40.2	35.9	35.0	33.8
564	55.5	52.6	43.0	40.2	37.5	34.1	33.0	32.0
566	73.5	70.9	58.0	55.3	49.5	47.8	43.2	42.8
Average deviation from (2)	+1.7		+1.5		+1.9		+0.4	
Maximum deviation from (2)	+3.9		+4.4		+4.3		+1.2	

This lack of agreement has little significance, however, as neither of the two methods are at present standard procedures. The results shown in these tables were obtained as a part of this investigation and are included principally to disclose the lack of agreement between these two methods and the pipette method. If all three methods were in agreement, no change from the standard value of L would be required.

TABLE 13.—*Comparison of analyses by (1) Bouyoucos hydrometer (L=0.70 HD), (2) specific gravity hydrometer (L=HD) and (3) pipette method (Bureau of Public Roads laboratory)*

Soil no.	Percentage finer than—											
	0.040 mm			0.005 mm			0.002 mm			0.001 mm		
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	1()	(2)	(3)
5782	97.8	97.1	97.2	80.6	80.2	83.5	56.2	53.7	59.0	35.3	32.0	39.6
5822	55.0	54.5	57.2	37.2	36.2	35.5	27.5	29.1	27.3	21.5	24.7	21.7
5833	65.0	63.7	67.3	24.7	25.4	24.0	15.1	16.0	14.6	9.5	10.2	9.7
5838	92.2	92.5	91.0	50.9	48.6	50.3	33.2	33.0	32.3	23.8	23.0	23.3
5842	64.1	60.5	70.6	30.2	32.3	30.9	24.2	25.4	25.2	19.8	20.7	18.7
5854	93.0	95.0	94.6	75.9	74.0	76.8	56.0	55.0	56.8	39.0	39.0	41.9
6062	28.3	28.5	29.2	10.2	12.5	9.8	7.4	9.0	6.0	5.6	5.9	3.7
6098	74.5	73.8	75.4	40.8	41.2	40.0	31.5	31.5	29.0	24.0	24.1	22.0
Average deviation from (3)	−1.4	−2.0		−0.1	−0.1		+0.1	+0.3		−0.3	−0.1	
tion from (3)	−1.4	−2.0		−0.1	−0.1		+0.1	+0.3		−0.3	−0.1	
Maximum deviation from (3)	−6.5	−10.1		−2.9	−3.3		−2.8	−5.3		−4.3	−7.6	

As a matter of fact, however, the significance of the difference of results furnished by the different methods is indicated by the different values of L as shown in tables 13 and 14.

TABLE 14.—*Comparison of analyses by (1) Bouyoucos hydrometer (L—0.49 HD) and (2) beaker method*

Soil no.	Percentage finer than—							
	0.020 mm		0.010 mm		0.007 mm		0.005 mm	
	(1)	(2)	(1)	(2)	(1)	2()	(1)	4 (2)
558	24.3	25.3	19.8	20.1	17.0	17.1	14.8	16.0
559	49.4	47.5	35.2	34.6	29.6	29.5	24.6	25.4
560	68.8	69.5	55.4	57.9	50.0	50.8	44.0	45.4
561	64.2	64.6	51.6	52.0	45.2	45.0	40.0	40.0
562	47.2	49.4	34.4	64.9	28.8	28.4	24.2	25.8
563	57.6	55.1	44.8	41.6	39.0	35.9	33.6	33.8
564	54.0	52.6	41.8	40.2	36.4	34.1	32.0	32.0
566	72.0	70.9	56.4	55.3	48.2	47.8	42.0	42.8
Average deviation from (2)	+0.3		+0.4		+0.7		−0.8	
Maximum deviation from (2)	+2.5		+3.2		+3.1		−1.6	

SUMMARY

However elaborate the method, no mechanical soil analysis based on Stokes's formula is capable of yielding an exact determination of the particle size distribution. In fact the results of such analyses are to a large extent dependent upon certain arbitrary conditions of test. A basis for estimating the relative accuracy furnished by the various test methods does not exist. Their efficiency must be determined upon the basis of the time and effort required for performing the tests and the scope of the data obtained.

In the hydrometer method, as in all other methods of mechanical soil analysis based on sedimentation of soil particles through water, thorough dispersion of the soil sample is desirable. The apparatus specified in the procedures, especially with the added improvements discussed in this report, is highly efficient for this purpose, and in conjunction with the supplementary treatments determined according to the plasticity of the soil, provides the necessary degree of dispersion with a minimum of time, attention, and effort.

A high degree of deflocculation or maintenance of the separation of the suspended particles is also of importance to the success of the test, and, with few exceptions, is adequately effected by use of the deflocculating agent specified in the procedures.

Readings of a sensitive hydrometer in soil suspensions serve very adequately to produce the data required for the determination of the grain size distribution in subgrade soils.

In the use of Stokes's formula as applied to the hydrometer test data, the values of L equal (1) to 0.42 of the total immersed depth of the Bouyoucos hydrometer, and (2) to the depth of the center of volume of the specific gravity hydrometer yield satisfactory results in the mechanical analysis of subgrade soils.

The variations of test conditions from those which the use of Stokes's formula assumes, combined with the fact that the value of L is not critical in many cases, may cause one to question the need of all the refinements included in the testing procedure as published.

For average soils tested under average conditions, the clay content based on the Bouyoucos hydrometer reading at 60 minutes, with hydrometer readings properly corrected for temperature and specific gravity and the fine clay content based on readings of the same hydrometer at 370 minutes are likely to be fairly accurate. If the specified refinements required appreciable time and effort, the desirability for making them for average soils would be open to serious question if only approximate results were wanted and the soil accumulation curve were not required.

These refinements are necessary in the analysis of unusual soils, however, and do increase the accuracy of tests on ordinary soils. They must, therefore, of necessity be included in any general procedure. Furthermore, by means of the graphical method of solution all of the corrections can be made without increasing noticeably the effort required to perform the tests.

The hydrometer method is not confined to the use of a hydrometer of any one type or calibration; the principal requirement is merely that the hydrometer shall be sensitive enough to measure the density of soil suspensions with an accuracy sufficient to give a reasonably close determination of the percentage of suspended soil. The relation between density of suspension and percentage of suspended soil expressed by equation 3 offers a basis for transforming the readings of any hydrometer into terms of soil percentage.

The depth of settlement, L, for use in Stokes's formula, which is likely to vary with different shapes of hydrometer bulbs, must be determined for any type of hydrometer other than those described in this report. This may be done on the basis of the relation between the hydrometer test data and the percentages of the different fractions furnished by some desired method of mechanical analysis or on the basis of results furnished by a hydrometer which has been standardized.

In spite of the fact that the depth of settlement, L, is a variable for which no one correction can be universally applied, the hydrometer test as used by the Bureau of Public Roads yields, with minimum effort, results which are in substantial agreement with those furnished by the more elaborate pipette method.

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EFFECT OF CURING CONDITIONS ON THE STRENGTH OF CEMENT MORTAR

Reported by D. O. WOOLF, Associate Materials Engineer, and K. F. SHIPPEY, Junior Highway Engineer, Division of Tests, United States Bureau of Public Roads

A TENDENCY for portland-cement-mortar specimens, particularly the standard 1:3 briquets, to retrogress in strength with age when stored in water has been noted many times. Since a corresponding retrogression in the strength of concrete specimens stored in moist air is rarely found, it has generally been assumed that the reduction in strength is not due to any defect in the cement but rather to some feature or treatment involved in the method of test. An explanation which has been advanced is that retrogression is due to the solution of a part of the cement by the storage water. If this is true, the reason for the decrease in strength of specimens stored in running water would be explained.

To determine if this theory is correct, an investigation of the effect of curing conditions on the strength of cement mortar was recently undertaken by the Bureau. Both briquets and 2-inch cubes were prepared with four different cements, including one high-early-strength cement. The briquets were made in accordance with American Society for Testing Materials Standard Method C 77–30, and the cubes were prepared essentially as described in a paper by E. M. Brickett, American Society for Testing Materials Proceedings, 1928, part II.

A mix of 1:2.75 by weight was used in the preparation of the cubes with a water-cement ratio of 0.53 by weight (0.80 by volume). The sand used is designated as "run-of-mine Ottawa sand" from Ottawa, Ill., and consists of the material from which standard Ottawa sand is obtained. Tests for consistency made on the 10-inch flow table using thirty ½-inch drops in 30 seconds gave a flow of approximately 90 percent with the above mixture. To insure uniform preparation of the test specimens, 54 cubes, or briquets, were molded in 1 day with a single cement. This furnished sufficient specimens for one series of tests at all ages with each condition of storage. After curing for 24 hours in the moist closet, each series of 54 specimens was separated at random into 3 sets of 18 specimens each. One set for each of the four cements was stored until date of test under each of the following conditions:

1. In running water
2. In still water
3. In moist air.

VARIATION OF STRENTH WITH AGE STUDIED

Tests were made at ages from 7 days to 1 year on both types of specimens cured under each of the above conditions of storage. All specimens were broken in a wet condition, the specimens cured in moist air being placed in still water for 24 hours immediately prior to test. Three series of specimens of each type were made with each of the normal portland cements, and two series with the high-early-strength cement, each series containing 54 specimens of each type.

The results of these tests are shown in the accompanying figures. Each point is the average of 9 individual tests in the case of the normal cements and 6 tests in the case of the high-early-strength cement.

The three normal portland cements exhibit in general the same characteristics. In the tension tests, the briquets stored in both running and still water show either retrogression or little change in strength after 28 or 90 days. Cement 3 shows a marked retrogression after 90 days, which continues to the age of 1 year. Cements 1 and 2 show retrogression after 28 days, this being most marked in the case of the running-water specimens for cement 1. The briquets cured in moist air show in two cases a steady increase in strength to the last testing period. In the case of cement 1, the moist-air specimens increase in strength to an age of 90 days, show a slight decrease in strength to an age of 270 days, and then recover strength at 1 year to exceed slightly the maximum obtained at an earlier period.

The tension specimens prepared with high-early-strength cement and cured in water or moist air develop retrogression after ages of 28 or 90 days. This is very marked for the briquets cured in both running and still water. The moist-air specimens show a small decrease in strength after 90 days and hold this strength to 1 year.

The compression tests of the normal portland cements show, as a general rule, an increase in strength to an age of 270 days for all wet storage conditions followed by little change in strength at 1 year. In the case of cement 3 the specimens cured in still water give some erratic results which cannot be explained at present. The compression tests made on the high-early-strength cement show a continual decrease in strength after 28 days for the specimens cured in running water, while the specimens cured in still water attain a maximum strength at 90 days and hold this strength at later periods. The specimens cured in moist air increase in strength to 180 days after which the strength is reduced considerably.

The results of these tests show that cement-mortar briquets used in the tensile strength test are usually subject to retrogression after ages of 28 days to 180 days when cured in water, either running or still. Briquets stored in moist air are usually found to show a steady increase in strength for ages up to 1 year, but the 7- and 28-day strengths are appreciably lower than those of the water-cured specimens. Briquets made with high-early-strength cement and stored in water or moist air show more marked retrogression in strength at an earlier age than is found for normal portland cement.

Little retrogression is found in the compression tests of normal portland-cement mortars. A definite retrogression in compressive strength, however, is found for high-early-strength cement specimens cured in running water, and those cured in still water fail to increase in strength after 90 days.

LOSS OF STRENGTH ATTRIBUTED TO SOLUTION OF PART OF CEMENT IN STORAGE WATER

The retrogression in strength found in small specimens of portland-cement mortar has been attributed to the solution and removal of a part of the cement by

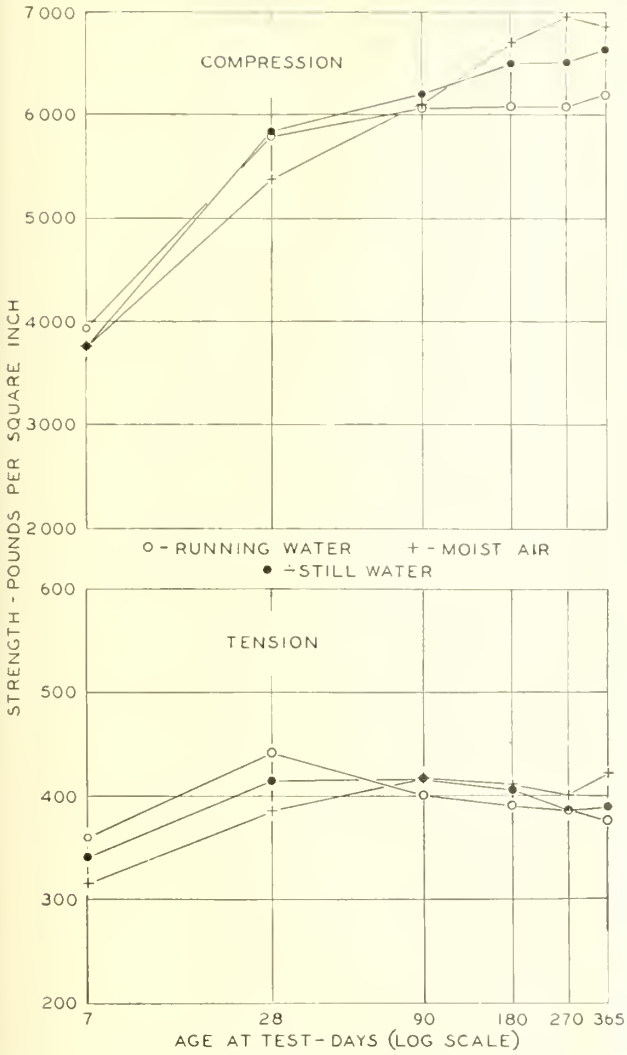

FIGURE 1.—STRENGTHS OF CUBES AND BRIQUETS MADE WITH NORMAL PORTLAND CEMENT NO. 1.

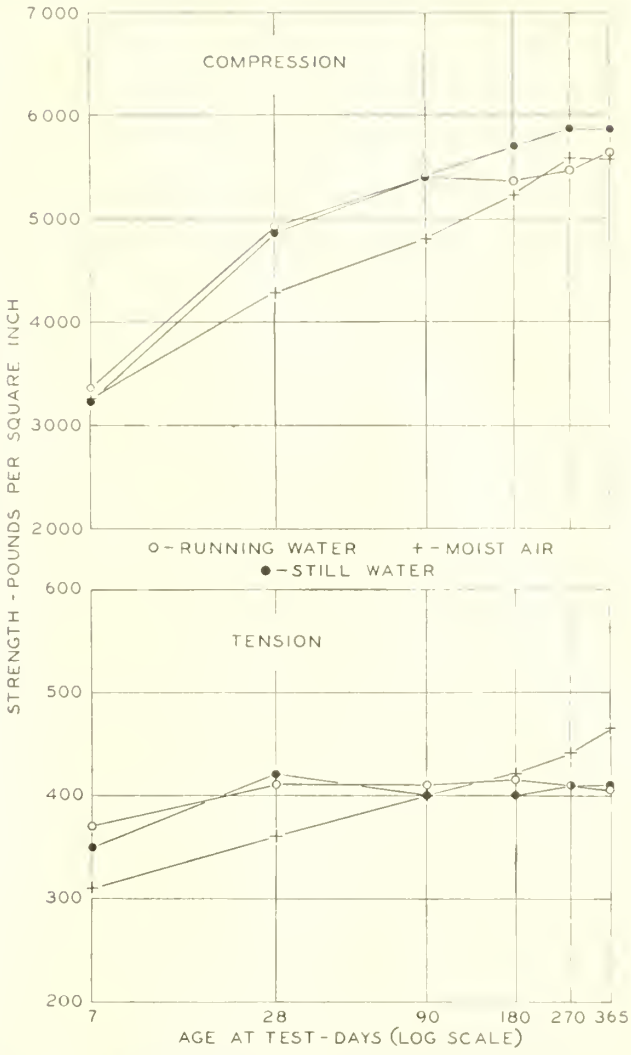

FIGURE 2.—STRENGTHS OF CUBES AND BRIQUETS MADE WITH NORMAL PORTLAND CEMENT NO. 2.

the storage water. This theory is believed to be substantiated by these tests. In this series of tests the conditions which might affect the strength were the same for all specimens except the medium in which they were stored. The occurrence of retrogression to the greatest extent in the water-cured specimens is indicative of the nature of the weakening action, that is, by solution of a portion of the cement. The still-water specimens were stored in an open pan, and frequent additions of water were necessary to replace that lost by evaporation. It is believed that the retrogression found in the still-water specimens would have been considerably smaller if the specimens had been stored in covered containers with a minimum of water. The fact that retrogression is found in the compression tests at a later date and to a lesser degree than in the tension tests is attributed to the difference in size and shape of the specimens used. If each type of specimen were subjected to the same solvent action of water, the briquet would be affected to the greater extent due to its smaller cross section.

TESTS FAIL TO CONFIRM THEORY ATTRIBUTING LOSS OF TENSILE STRENGTH TO UNEQUAL STRESS DISTRIBUTION IN BRIQUET

In the first report of the Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement [1] the retrogression of strength found in cement mortar briquets is attributed to the inequality of stresses set up in a briquet by the method of loading coupled with the rigidity of the specimen. The load applied at four points by the clips must be transmitted in a very short distance to the entire cross section of the briquet and it is claimed that a uniform distribution of this stress over the minimum section is impossible. Prof. J. B. Johnson concluded from an analytical study of the problem that the maximum stress in a briquet of

[1] Committee C–1, A.S.T.M., Data Book, published by the committee, section II, p. 74, July 1919.

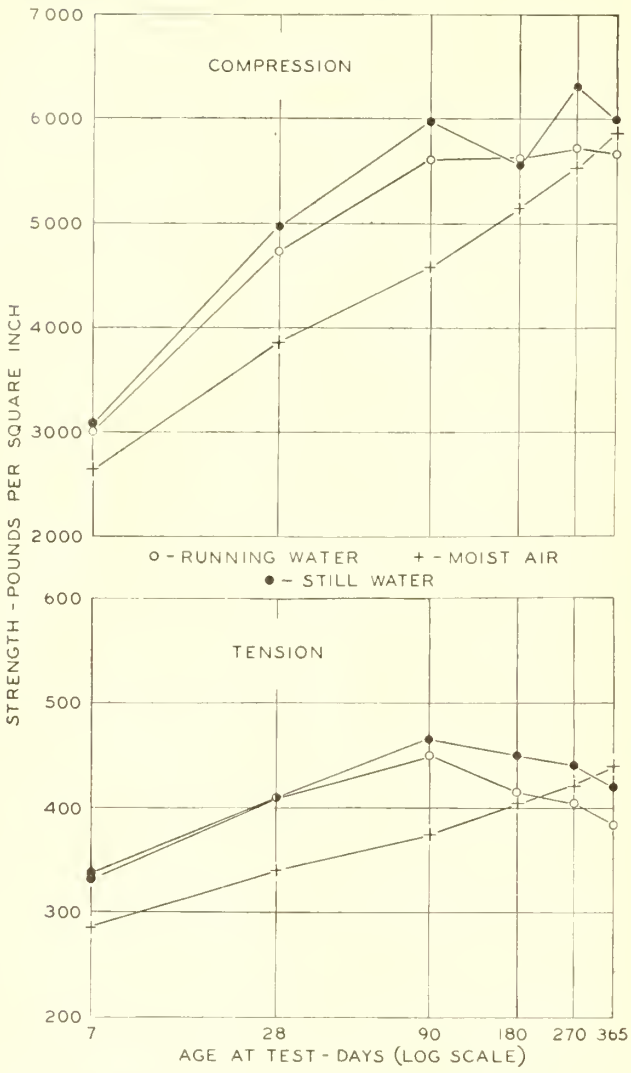

FIGURE 3.—STRENGTHS OF CUBES AND BRIQUETS MADE WITH NORMAL PORTLAND CEMENT NO. 3.

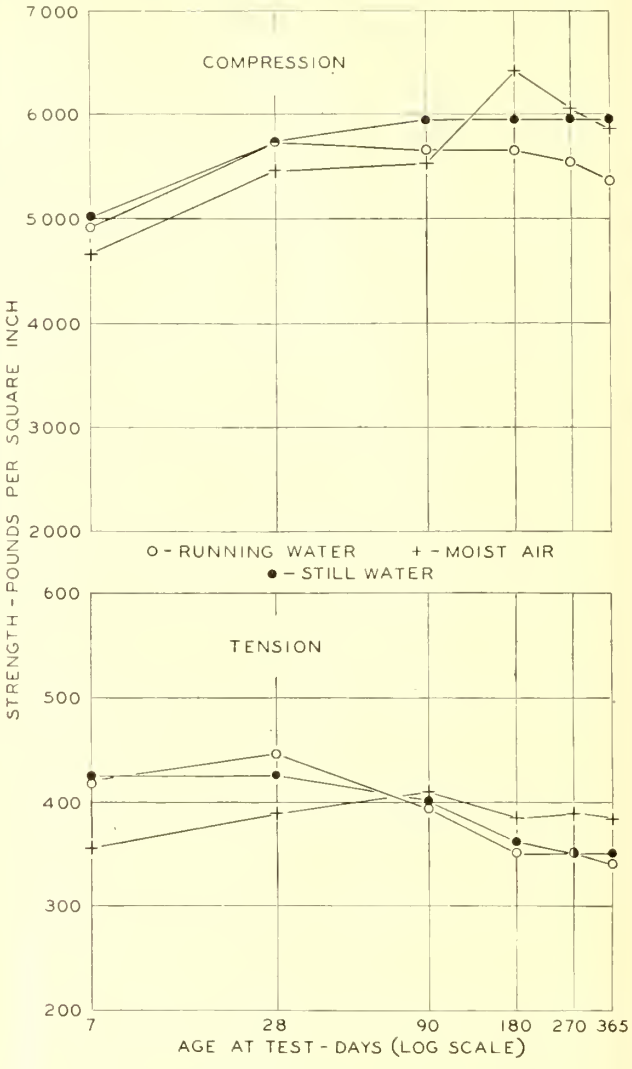

FIGURE 4.—STRENGTHS OF CUBES AND BRIQUETS MADE WITH HIGH-EARLY-STRENGTH CEMENT NO. 4.

the form now used is about 1.54 times the minimum stress. E. G. Coker made an experimental determination of stresses in briquets, using the effect of polarized light passing through models of transparent celluloid, and concluded that the highest stress in the minimum section of the test specimen is 1.75 times the mean stress. With a very rigid material the opportunity for an adjustment of stresses within the specimen is much less than in a material of low rigidity. According to this theory a very high stress is developed in the older briquets at the edges of the section during the first stages of loading, and a tearing action is produced which results in rupture at a lower stress than would have been withstood at an earlier age when the lower rigidity permitted of a more uniform distribution of stress.

The results obtained in this investigation do not seem to be in agreement with the above theory. The briquets cured in moist air show a steady increase in strength. According to the theory, the specimens at age of 270 or 365 days should fail at a lower load than the 7- or 28-day specimens. Such is not found and the conclusion is reached that this theory is incorrect.

TESTS INDICATE STANDARD CURING PRACTICE SATISFACTORY

The present standard specifications for testing portland cement require that the specimens be cured in water. These tests, involving a limited number of observations, do not show any reason for changing from the standard practice of curing mortar specimens for ages up to and including 28 days. Little preference is seen between running- and still-water storage, as specimens stored under either condition develop essentially the same strength at 7 or 28 days. The moist-air storage is, however, recommended for tests of mortar specimens at greater ages.

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

JULY 31,1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage	MILEAGE Total		
		$	$					$	$				$	
Alabama	2,355.6	4,742,320.50	2,371,160.14	86	119.9	110.2	230.1						3,199,828.75	Alabama
Arizona	1,280.8	2,820,591.24	1,320,741.91	93	57.7	126.8	184.5	59,134.40	31,023.48		6.1	6.1		Arizona
Arkansas	1,945.1	4,339,982.17	2,055,467.95	79	118.8	87.4	206.2	1,227,502.40	613,567.11	102.0	16.0	118.0	661,199.03	Arkansas
California	2,534.0	6,407,652.46	1,523,062.16	85	126.4	32.1	158.5	73,760.41	17,153.65	.9	2.0	2.9		California
Colorado	1,892.3	2,310,126.95	1,040,247.34	84	85.3	11.5	96.8	1,350,432.71	543,285.50	40.6	52.2	92.8	54,574.57	Colorado
Connecticut	299.5	4,037,363.34	1,659,219.49	83	41.0	5.3	46.3						183,167.11	Connecticut
Delaware	405.9	309,823.50	59,812.72	98	7.4	7.2	14.6							Delaware
Florida	663.6	5,425,056.85	2,576,211.42	81	158.0	11.5	169.5						1,008,446.01	Florida
Georgia	3,307.2	3,163,588.65	1,292,758.81	91	89.9	129.8	219.7						100,636.93	Georgia
Idaho	1,597.1	2,381,148.88	859,613.66	83	85.3	113.9	199.2	138,413.39	66,374.60	5.7	26.8	32.5	40,793.01	Idaho
Illinois	3,143.7	19,142,650.18	7,158,083.12	78	590.9	60.6	651.5	587,949.56	290,001.86	20.5		20.5	95,967.25	Illinois
Indiana	2,122.1	8,169,261.24	2,932,592.03	88	278.2	21.9	300.1	609,598.98	98,177.17	20.1	3.0	23.1		Indiana
Iowa	3,677.4	2,878,157.82	343,197.16	90	140.2	14.3	154.5							Iowa
Kansas	4,071.5	3,260,041.34	933,948.55	74	187.7	50.4	238.1	299,371.96	106,403.33	89.9	13.8	103.7	27,848.05	Kansas
Kentucky	1,969.2	3,895,175.12	1,402,517.10	79	157.5	97.0	254.5	104,198.71	46,331.37	2.4	8.7	11.1	15,535.98	Kentucky
Louisiana	1,622.9	6,281,251.44	2,735,866.90	71	38.9	24.8	63.7						4,190.58	Louisiana
Maine	832.3	1,817,938.30	450,411.28	74	55.2	2.2	57.4							Maine
Maryland	874.1	830,173.77	93,860.77	91	32.0	1.5	33.5	243,278.40	25,673.37	5.4		5.4		Maryland
Massachusetts	876.4	4,115,901.96	972,213.48	83	57.2	4.9	62.1						399,016.29	Massachusetts
Michigan	2,416.2	5,632,625.42	2,119,097.45	80	253.8	71.7	325.5	141,300.00	50,705.00	11.7		11.7	198,533.35	Michigan
Minnesota	4,309.6	4,759,578.15	206,528.91	98	151.0	139.9	290.9						30,504.16	Minnesota
Mississippi	1,862.5	6,539,603.92	3,245,253.58	73	179.4	112.9	292.3	81,968.38	40,984.17	3.2	3.4	6.6	2,791,531.23	Mississippi
Missouri	3,262.1	3,951,429.70	681,209.60	54	141.9	27.4	169.3	193,215.66	61,048.60	5.8	6.4	12.2		Missouri
Montana	2,951.6	6,171,603.82	3,442,620.60	89	414.5	246.1	660.6	197,566.09	111,137.88	.5	57.0	57.5	95,837.70	Montana
Nebraska	4,284.5	5,696,135.19	2,663,872.42	92	138.1	152.7	290.8						56,616.07	Nebraska
Nevada	1,344.2	2,083,553.46	658,660.73	97	20.7	152.9	173.6	159,568.45	79,784.22	9.9	33.9	43.8	61,164.41	Nevada
New Hampshire	450.7	536,710.27	214,442.76	81	11.4	.5	11.9	20,531.40	8,212.56	.4		.4	77,387.63	New Hampshire
New Jersey	637.9	5,309,569.73	1,735,318.97	93	55.6		55.6						138,625.11	New Jersey
New Mexico	2,456.7	873,698.68	345,122.59	92	40.3	35.3	75.6	72,488.06	39,868.43	7.2	1.6	8.8	88,572.49	New Mexico
New York	3,524.5	18,013,830.05	5,483,100.00	66	471.3	32.0	503.3						355,781.37	New York
North Carolina	2,379.9	4,941,170.70	2,477,828.15	82	492.1	28.1	520.2	247,056.89	123,528.41	43.0	.3	43.3	977,154.06	North Carolina
North Dakota	5,525.5	3,344,326.20	1,483,434.12	59	225.2	342.3	567.5	431,095.31	209,476.42	31.1	112.8	143.9	244,263.71	North Dakota
Ohio	3,094.4	7,020,159.85	1,794,804.93	83	142.4	51.7	194.1	401,335.00	154,207.94	7.8	.3	8.1	43,622.92	Ohio
Oklahoma	2,539.1	3,587,419.84	1,137,922.90	88	165.0	67.1	232.1	962,586.25	316,197.29	67.6	5.6	73.2	215,224.61	Oklahoma
Oregon	1,642.7	3,812,878.93	1,572,517.58	85	97.8	69.1	166.9	35,805.00	18,486.11	1.4		1.4	34,012.84	Oregon
Pennsylvania	3,321.4	8,835,667.49	2,447,528.83	83	304.1	14.0	318.1						4,331.54	Pennsylvania
Rhode Island	277.5	809,913.11	283,007.01	84	16.6	4.5	21.1						70,834.03	Rhode Island
South Carolina	1,962.5	3,419,613.38	1,280,542.73	93	161.2	154.7	315.9						9,543.92	South Carolina
South Dakota	4,304.1	3,222,473.45	1,315,670.92	90	261.6	202.8	464.4	211,412.66	93,100.65	8.1	27.9	36.0	169,101.27	South Dakota
Tennessee	1,784.1	3,692,462.69	1,845,510.43	80	115.0	24.8	139.8						576,790.79	Tennessee
Texas	8,237.5	13,607,283.12	4,331,725.34	85	494.3	425.5	919.8	649,849.26	180,684.47	22.7	11.7	34.4	4,313.20	Texas
Utah	1,317.2	1,751,085.18	738,800.09	87	111.7	26.6	138.3	246,126.94	104,802.54	19.6	12.5	32.1	730.62	Utah
Vermont	391.1	465,379.08	73,902.58	41	22.4	6.6	29.0	19,375.18	850.98	.3		.3	238.42	Vermont
Virginia	2,029.8	3,577,897.84	1,576,503.95	88	146.3	50.5	196.8	80,190.16	22,057.12	4.1		4.1	176,551.34	Virginia
Washington	1,340.2	2,891,680.79	849,320.63	83	100.0	9.7	109.7	223,273.36	34,000.00		8.0	8.0	3,356.82	Washington
West Virginia	933.0	2,676,604.45	1,100,437.50	83	99.8	14.7	114.5						18,882.07	West Virginia
Wisconsin	2,764.0	5,315,322.60	1,254,767.95	81	132.4	101.9	234.3	52,061.50	21,500.00		.9	.9		Wisconsin
Wyoming	2,188.3	2,528,229.84	858,695.48	97	207.2	137.9	345.1						153,262.89	Wyoming
Hawaii	121.5	1,056,210.46	840,299.35	53	25.1		25.1	87,664.50	29,902.55	.7		.7	485,620.83	Hawaii
TOTALS	109,124.8	222,452,313.10	79,844,439.07	81	7,625.7	3,617.2	11,242.9	9,207,913.97	3,538,526.78	532.6	410.9	943.5	12,874,192.06	TOTALS
Construction completed		178,046,000	65,063,000											
Balance uncompleted		44,406,000	14,781,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid in the initial improvement. In general, such additional work consists of the construction of a surface of higher type than was provided

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 7 SEPTEMBER 1933

REFLECTING BUTTONS ON HIGHWAY SIGNS ADD TO SAFETY OF NIGHT DRIVING

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Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 7 September 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

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THE COMPARATIVE VISIBILITY OF STANDARD LUMINOUS AND NONLUMINOUS HIGHWAY SIGNS

REPORT OF INVESTIGATIONS BY THE UNITED STATES BUREAU OF PUBLIC ROADS, IN COOPERATION WITH THE NATIONAL BUREAU OF STANDARDS

Prepared by F. W. MILLS, Highway Engineer, Division of Highway Transport, U.S. Bureau of Public Roads

THE REPORT of the subcommittee on traffic control and safety of the American Association of State Highway Officials, rendered at the annual meeting held in Pittsburgh, November 17 to 20, 1930, included the following recommendations:

STANDARDIZATION OF LUMINOUS SIGNS

Since it is uncertain what color and size of reflector buttons and color of background should be used on the reflector signs when made as a combination day and night sign, and due to the variable focal length of buttons that may be used on these signs, it is recommended that the Bureau of Public Roads consult with the Bureau of Standards in conducting a series of tests to learn of the most appropriate combination to use with the view of making any necessary changes in the present designs. One of the most

FIGURE 1.—OBSERVER'S STATION AT EYEPIECE OF TACHISTOSCOPE.

important questions is the color of the background most appropriate for use with reflector buttons, keeping in mind both day and night visibility. For instance, is a sign with reflector buttons set in a black panel on a white background more efficient than one having buttons in a white panel on a black background, or is some other combination of color better? Also, the focal length and size of buttons as well as the number of buttons for a specific size of letter should be determined.

At the annual meeting of the American Association of State Highway Officials, held on September 28, 1931, it was resolved that:

The executive committee of the American Association of State Highway Officials appoint a special sign committee to act as a subcommittee of the regular association committee on traffic control and safety, and that this sign committee be made generally representative over all parts of the country so far as possible in a group of not more than five members; that the members of this committee shall be drawn preferably from those members who are directly in contact and responsible for the installation and maintenance of highway signs and markers.

FIGURE 2.—PENDULUM FOR REGULATING TIME OF OBSERVATION.

The executive and publicity committee of the National Conference on Street and Highway Safety, at a meeting in New York City on June 23, 1931, adopted the following:

After further discussion it was voted that the executive committee of the National Conference on Street and Highway Safety favors the combination of the two codes in one publication and the setting up of a joint committee representing the American Association of State Highway Officials and the conference to formulate such a combined code and that Colonel Barber represent the conference in dealing with the highway group in the appointment committee.

Pursuant to these resolutions, a joint committee consisting of eight members representing the National Conference on Street and Highway Safety and seven members representing the American Association of State Highway Officials was created and the first meeting was held in Washington on March 24, 1932.

In harmony with the previous recommendation of the subcommittee on traffic control and safety, the joint committee was authorized: (1) To make such revisions as may seem necessary to harmonize or combine the

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two codes; (2) to make such additions as will produce a single and complete code for both rural and municipal use and to complete, in cooperation with the Bureau of Public Roads and Bureau of Standards, investigations recently inaugurated covering the use of reflecting elements; and (3) to introduce into the series of standard signs such other signals as may be indicated by the results of the investigation.

In accordance with the second item of this authorization the project covered by this report was inaugurated, to be carried out under the leadership of Drs. J. Franklin Meyer and M. G. Lloyd for the Bureau of Standards and Mr. F. W. Mills for the Bureau of Public Roads.

FIGURE 3.—EQUIPMENT FOR NIGHT OBSERVATIONS.

Broadly considered, the scope of the investigations covered three principal phases: First, personal observations of standard signs having different color combinations as between background and legend, letters of various standard sizes without reflector buttons being used for this purpose; second, night observations of signs equipped with reflector buttons; and third, daylight observations made on the same signs to determine what effect there is, if any, on legibility, thus furnishing a basis for comparing their legibility with that of the signs not equipped with reflector buttons. The tests extended over a period that included summer as well as winter climatic conditions and offered an opportunity to view the signs in bright sunlight, during rains, snow, and fog, and against the changing natural background.

TACHISTOSCOPE DEVISED FOR REGULATING TIME OF OBSERVATION

Previous investigations in this field apparently did not consider the essential element of personal acuity as registered by time intervals. It therefore devolved upon the Bureau of Standards to design an apparatus by means of which the periods of free observation could be accurately measured and the time interval changed at will. Several general schemes were considered, such as the use of shutters similar to camera diaphragms, but these were lacking in simplicity and portability, or restricted the field of vision in a greater or less degree. A tachistoscope, meeting these requirements, was finally designed at the Bureau of Standards. It consisted of two major units, (*a*) a screened eyepiece with two shutters moving in parallel vertical planes, and (*b*) an A frame or easel supporting a pendulum by which the time interval is regulated and controlled. The screened eyepiece or observer's window is set up on an ordinary tripod with adjustable legs and carries the two shutters in front of an eye shade. The shutters are held in place by magnets. Figure 1 shows an observer seated before the eyepiece of the tachistoscope.

The A frame supporting the pendulum is approximately 5 feet high and 4½ feet across the base. It is held vertical by a hinged strut at the back, thus forming a complete tripod. The front of the frame carries a 5- by 4-foot weatherproofed sheet of wall board reinforced with battens to prevent warping. A 3- by 4- by 6-inch block situated at the top of the frame and protruding through the wall board provides a support and carrier for a pendulum one meter in length. The pendulum rod extends 25 centimeters below the bob so that in swinging it operates contacts which open the circuits of the two magnets at the observing window and releases the shutters. These contacts are carried on brass brackets affixed to the face of the wall board in the arc described by the pendulum. The contacts consist of a simple spring arrangement so constructed that when touched by a swinging pendulum the circuit is opened. One contact controls the opening shutter and the other the closing shutter of the observer's window. The contacts are attached to the bracket by a thumb screw and can be adjusted to any desired time interval from 0.1 to 1.0 second or more. Figure 2 shows the pendulum and contacts.

In order to minimize or entirely overcome the element of surprise, the switch controlling the movement of the pendulum, and consequently of the shutters, is operated by the observer. It was early recognized that the observer must also become used to the noise due to shutter movement; accordingly, in each case the apparatus was operated several times by the observer until all involuntary winking or other distraction was overcome and tolerance established. It was also foreseen that in the course of a long series of observations the sequence in which the signs were displayed might become known to the observer and the reading obtained be by memory or by suggestion. This possibility was eliminated by reversing the order of display and sequence of changes in the signs and also the time periods during which they were observed.

It is believed that the investigation has been as exhaustive as possible and that the results obtained lead to definite conclusions regarding relative efficiency within the scope permitted.

For night operations, two standard automobile headlights were mounted in front of the observation screen. A small telescope with cross hairs, by means of which the light beams could be directed so as to fall upon the signs, was attached to the cross arm between the head lights (fig. 3).

In choosing the field for the operations, great care was taken in selecting a section of roadway which would present the average background encountered on the average highway. The main driveway at the Bureau of Standards in a large measure met this consideration. It runs directly east and west, so that the sunlight was directly behind the sign during the morning hours and fell directly on the sign in the late afternoon with all penumbral phases during the day. Morning

shadows were thus compensated for by afternoon sunlight. The roadway was of concrete near the sign in the far distance and of asphalt in the observer's immediate foreground. A low concrete retaining wall and a concrete building were on the observer's left and brick buildings and vegetation immediately in the rear and to the right. Observations were taken at 200, 300, 350, 400, and 500 feet. Figures 4 and 5 show the general appearance of the area in which the tests were made.

In selecting the observers, an attempt was made to obtain a complete cross section of actual and potential automobile drivers. Many observers were obtained from the local employment agency through the interested cooperation of the American Automobile Association. The majority held drivers' permits but care was taken to select a due proportion of nondrivers. For the daylight operations, approximately 28 percent were female and 72 percent male and ages ranged from 17 to 70 years. Three observers claimed to be colorblind but were not exeluded on that aecount.

DAYLIGHT OBSERVATIONS ON NONLUMINOUS SIGNS

The nonrefleeting signs used for daylight observations were of three shapes with standard dimensions (24-ineh diameters): (1) Octagon; (2) square; and (3) square with diagonals vertieal (diamond). The ground colors were standard Federal yellow, black, and white. Black letters were used on the yellow and white baekgrounds and white letters on the blaek backgrounds. All letters were of standard height and stroke. In making up the various legends, separate letters were painted on heavy bristol board and attached to the signs as needed. This permitted transposition of the same letters or the use of different-sized letters on the same sign to check the accuracy of the previous reading.

All observations were made on signs of standard dimensions and design, adopted by the American Association of State Highway Officials. The hue and shade of the paint used for ground color on the yellow signs conform to the colorimetric requirements of rule 53 of the sectional committee on colors for traffic signals of the American Standards Association. These requirements have been adopted by the American Association of State Highway Officials as standard specifications and the shade is informally known as "Federal yellow." The three associated elements of sign recognition, that is, shape, color, and legend, were reported on and recorded separately.

A 3-way telephone system connecting the recorder at the observation button with the operator at the pendulum and the manipulator making the sign changes was installed. The recorder would indicate to the pendulum operator the time interval required and to the sign manipulator the desired change of the sign. In order that the observer should not be apprised of the character of the change or of the time interval, this information was conveyed either in code or by a previously arranged sequence of operation. When the proper adjustments had been made, the observer pressed a push button releasing the shutters. He was then required to inform the recorder whether or not a sign had been seen during the interval of free sight and, if so, its shape, color, and legend. The sign or its legend was then changed. The change might consist of a transposition of letters or the display of an entirely different sign. The transposition of letters or legend was particularly interesting in that it is apparent that if an observer could not read a 6-inch letter in a 0.5-second interval, it might be possible and frequently was so to read an 8-inch letter in the same interval and at the same distance. Obviously the large direction arrow might assist or even suggest recognition of the CURVE sign, but if the CURVE is changed to TURN and a large R or L substituted for the arrow, it will serve as a check on the accuracy of the previous read-

SUPERIORITY OF YELLOW BACKGROUND FOR DAYLIGHT VISIBILITY DEMONSTRATED; THREE-FOURTHS-INCH REFLECTING BUTTON FOUND EFFICIENT FOR NIGHT DRIVING

The findings in this investigation of standard luminous and nonluminous highway signs are briefly summarized as follows:

***1. Daylight observations on nonluminous signs.*—All evidence points to the definite conclusion that the standard yellow background with black letters or design, under all conditions reasonably to be expected in either urban or rural driving, is much superior to black on white or white on black. In addition to the greater readability of the legend the yellow background has a greater signal value, being more arresting to the average observer and more conspicuous by contrast with the average natural or artificial background. By the adoption of a definite shade and hue it should be possible to prohibit the use of this distinctive shade for any roadside advertisements or signs other than official highway signs.**

***2. Night observations on nonluminous signs.*—While these observations were not in any way conclusive, the inadequacy of daylight signs for night driving was clearly indicated. The effect of a snow background was especially noted.**

***3. Night observations on luminous signs.*—Observations were made with three sizes of reflector button, 0.95-inch, 0.76-inch, and 0.58-inch, under conditions simulating both rural and city driving. The conclusion was reached that the colorless button with a diameter of 0.76-inch and a minimum spacing between buttons of 1 inch, center to center, is, in general, the most efficient. For rural use the 0.58-inch button has a superior long-range signal value, but is less readable than the unit of larger diameter.**

***4. Daylight observations on luminous signs.*—The effect on distinctness in daylight of inserting reflecting buttons in the letters is not serious at a distance of 200 feet, even though approximately 50 percent of the area of the letter is taken up by colorless buttons. The difference in the effect on black and on white letters is negligible.**

***5. Recognition by shape of sign.*—The practice of outlining highway signs with reflector buttons, so that the road user may recognize a potential hazard at a distance by the distinctive shape of the sign plaque alone, is worthy of further development. The use of symbols, except for the arrows indicating curve or direction, is not considered as effective as outlining the sign with buttons.**

ing, the color, shape, and time period not having been changed.

For the tests of nonreflecting signs 70 observers were used who made a total of 4,563 observations at the various distances from 200 to 500 feet. The time intervals for legibility of the legend ranged from 1.0 to 0.6 second. This apparently was the shortest interval in which the legends could be distinctly read, the readability, of course, diminishing with the distance. The time intervals 0.5, 0.4, and 0.3 second were included to determine whether the observer could still see a sign at all and, if so, its shape, color and legend.

The contention that when two colors are used the background should be of the darker color is not in this instance conclusively borne out because the superiority of white on black over black with yellow background obtains only at 200 feet. One explanation of the apparent difficulty in reading black on white is that sun glare on the high gloss white background obscures the black letters and this is apparently verified when it is noted that the percentages of difference decrease appreciably as the distance is increased. At 400 feet both black and white combinations are equal and at 500 feet there is but 1 percent in favor of the

FIGURE 4.—GENERAL VIEW OF MAIN DRIVEWAY AT BUREAU OF STANDARDS, WITH OBSERVER'S STATION IN FOREGROUND.

AVERAGES OF ALL OBSERVATIONS INDICATE SUPERIORITY OF BLACK ON YELLOW

The average of the observations at all distances and for all time intervals establishes the following relative values: Readings of the black on white combination were 57 percent correct; of the black on yellow combination 66 percent correct, and of the white on black combination 59 percent correct. Figure 6 shows the variation in the percentage of correct readings with the distance of the sign from observer's station. The data are given in tabular form in table 1. Figure 7 gives the percentage of correct readings for each of the various signs and time intervals used, and for each of the distances from the observer.

In analyzing this tabulation, it is interesting to note that at 200 feet, white on black, i.e., white letters on black background, had a distinct advantage over black on white and a slight superiority over black on yellow. At 300 feet the superiority is to a small extent reversed with respect to the black and white combinations, being 2 percent in favor of the black on white, but at 350 feet it is again 5 percent the other way.

white on black. At all distances, except 200 feet however, black on yellow has the advantage of from 5 percent to 12 percent over either of the other combinations.

TABLE 1.—*Percentage of correct readings at each station, averaged for all time intervals*

Distance from observer	Color combination	Percentage of correct readings
Feet		
200	Black on white	8
	Black on yellow	9
	White on black	9
300	Black on white	6
	Black on yellow	7
	White on black	6
350	Black on white	5
	Black on yellow	7
	White on black	6
400	Black on white	3
	Black on yellow	4
	White on black	3
500	Black on white	3
	Black on yellow	4
	White on black	3

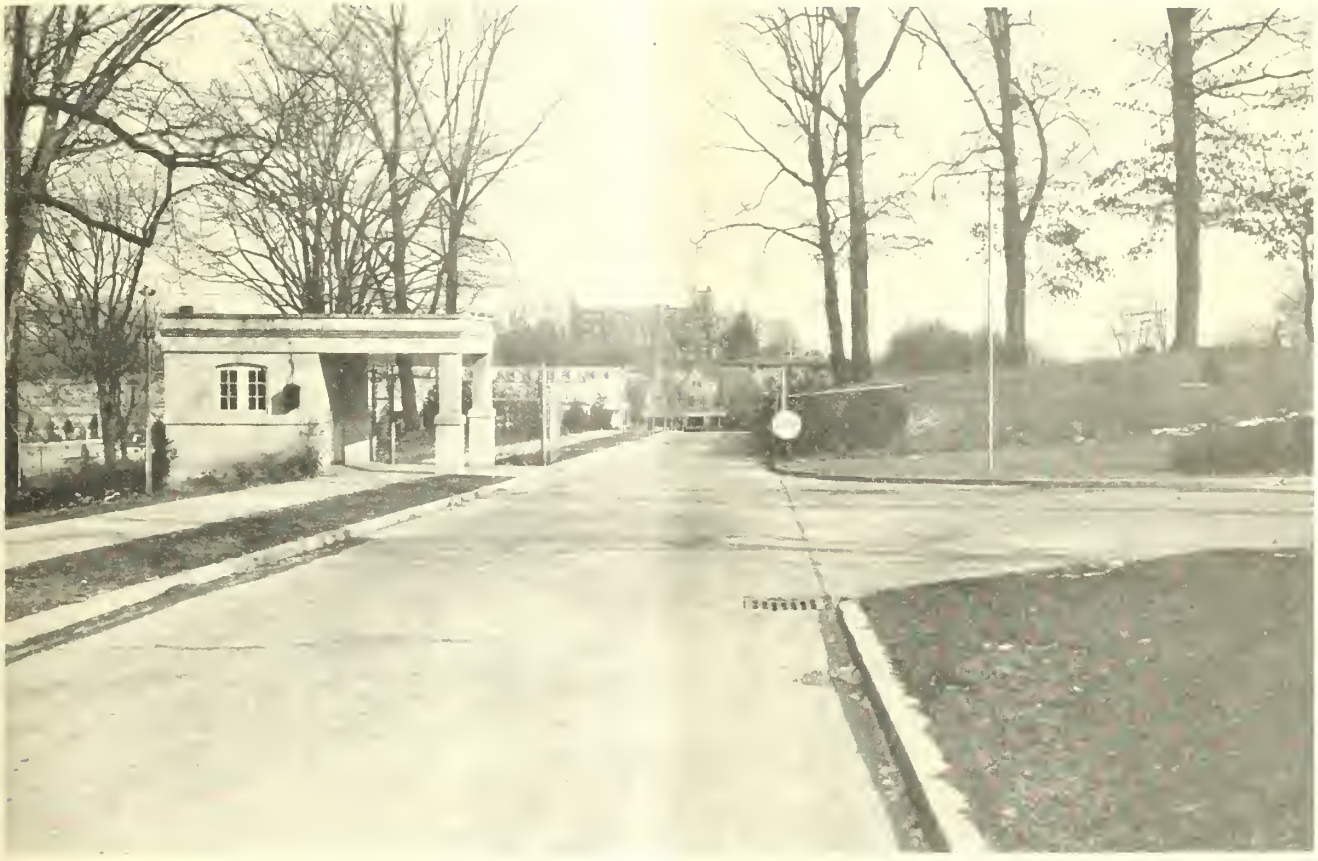

FIGURE 5.—WINTER VIEW FROM OBSERVER'S STATION.

DATA ON MEAN AND EXTREME DISTANCES ANALYZED

The previous discussion deals only with general averages of correct recognition of the signs, including all transpositions of the legend and time periods down to 0.6 second. A more complete analysis of the correct percentages obtained at the mid or critical distance and at the extreme stations (200 and 500 feet) follows. The data obtained with signs placed at these stations are given in tables 2 and 3.

The 350-foot distance apparently is the critical point beyond which there is an abrupt falling off in readability of the signs. At this station, 17 observers (6 female and 11 male) made 777 observations. Separating these according to time intervals and observing the SLOW sign (letters 8 inches in height with $1\frac{3}{16}$ inches stroke) at 1.0 second, we find that the black on white shows 78 percent, black on yellow 83 percent, and white on black 72 percent correct. The STOP sign (6-inch letters) at 0.8 second shows: Black on white 64 percent,

TABLE 2.—*Comparative visibility at mean distance (350-foot station)*

Sign	Size of letter	Time interval	Color combination	Percentage of correct readings
		Seconds		
SLOW	8 by $1\frac{3}{16}$ inches	1.0	Black on white	78
			Black on yellow	83
			White on black	72
STOP	6 by 1 inch	.8	Black on white	64
			Black on yellow	73
			White on black	58
CURVE	6 by $\frac{7}{8}$ inches	.7	Black on white	62
			Black on yellow	74
			White on black	57
ZONE	5 by $\frac{5}{8}$ inches	.6	Black on white	62
			Black on yellow	71
			White on black	50
SLOW	8 by $1\frac{3}{16}$ inches	.6	Black on white	70
			Black on yellow	77
			White on black	61

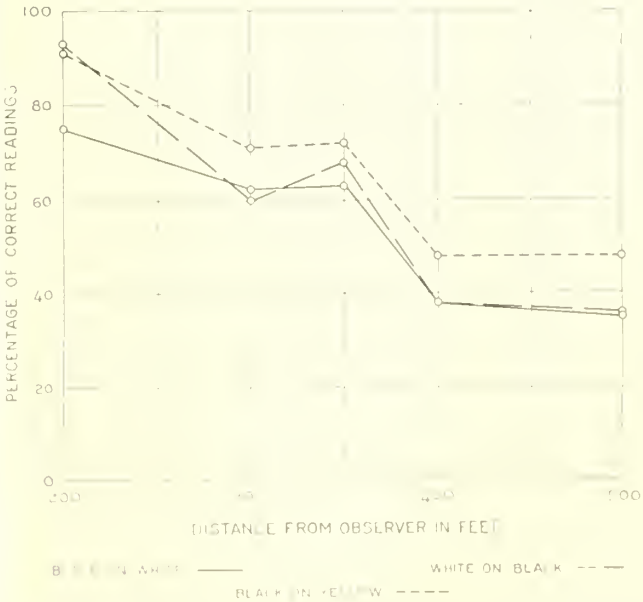

FIGURE 6.—PERCENTAGE OF CORRECT READINGS OF NONLUMINOUS SIGNS IN DAYLIGHT AT VARIOUS DISTANCES FROM OBSERVER. AVERAGE FOR ALL TIME INTERVALS AT EACH STATION.

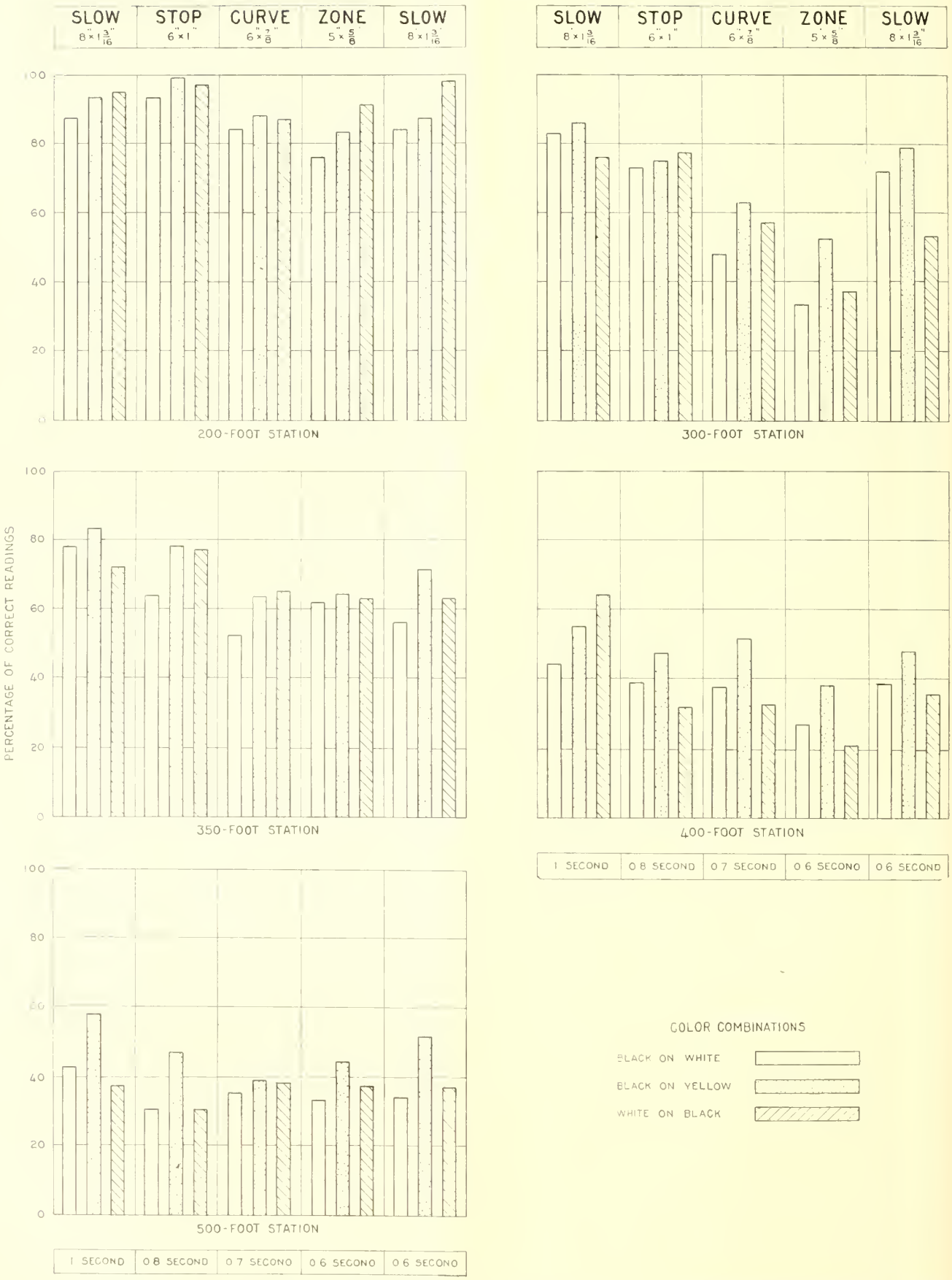

FIGURE 7.—PERCENTAGE OF CORRECT READINGS OF NONLUMINOUS SIGNS IN DAYLIGHT AT VARIOUS DISTANCES FROM OBSERVER AND FOR VARIOUS LENGTHS OF TIME OF OBSERVATION.

black on yellow 78 percent, and white on black 77 percent correct. The CURVE sign (letters 6 inches in height with ⅞-inch stroke) at 0.7 second shows: Black on white 52 percent, black on yellow 64 percent, and white on black 65 percent. With the SCHOOL ZONE sign (letters 5 inches in height and ¾-inch stroke) at 0.6-second interval, the percentages are: Black on white 62 percent, black on yellow 64 percent, and white on black 63 percent correct.

At this point the accuracy of the observer's reading was checked by again using the 8-inch letters on the SLOW sign. Except for the black on yellow the percentages obtained do not verify anticipations of increased legibility for the larger letter, for at the same time interval (0.6 second) the readings were: Black on white 56 percent, black on yellow 71 percent, and white on black 63 percent correct.

As in the general averages previously referred to, black on yellow still maintains its first place with white on black and black on white following in that order, the percentages at this station being 62 percent for black on white, 72 percent for black on yellow, and 67 percent for white on black.

The data are not broken down into time intervals showing the relative percipiency as between male and female drivers but the arithmetical average obtained at all intervals may be enlightening. The 6 females made 297 observations and the 11 males made 480 observations at the critical station (350 feet) with the following results:

FEMALE OBSERVERS

	Percent correct
Black on white	47
Black on yellow	64
White on black	57

MALE OBSERVERS

	Percent correct
Black on white	76
Black on yellow	79
White on black	77

Possibly the relatively low female average can be attributed to the fact that, of the 6 observers, 3 were nondrivers and therefore presumably unfamiliar with the signs. All of the 11 males, on the other hand, held driving permits.

At the extreme distances, the following results were obtained: At 200 feet the minimum distance, a total of 684 observations were made by 12 observers, 3 female and 9 male. The time intervals were 1.0, 0.8, 0.7, and 0.6 second and the sign copy as before. At 1 second, black on white registered 87 percent, black on yellow 93 percent, and white on black 95 percent correct. At 0.8 second, black on white registered 93 percent, black on yellow 99 percent, and white on black 97 percent correct. At 0.7 second, correct registration for black on white was 84 percent, black on yellow 88 percent, and white on black 87 percent. At 0.6 second, with the 5-inch letter, black on white registered 76 percent; black on yellow 83 percent, and white on black 91 percent correct. Again at 0.6 second, with the 8-inch letter, a marked increase in the correct readings is noted, giving for black on white 84 percent, black on yellow 87 percent, and white on black 98 percent correct. The difference in percentages in favor of the white on black as against the black on white is again pronounced, and white on black is superior also to black on yellow, except for the 6-inch letter at 0.8 second and the 5-inch letter at 0.7 second. In comparing again the visual acuity of the females with that of the males, quite the reverse of the condition at 350 feet is noted, in that for all the time intervals the females registered 90 percent for black on white; 93 percent for black on yellow, and 95 percent for white on black, as against 81 percent, 88 percent and 91 percent, respectively, for the males.

TABLE 3.—*Comparative visibility at extreme distances*

200-FOOT STATION

Sign	Size of letter	Time interval	Color combination	Percentage of correct readings
		Seconds		
SLOW	8 by 1 3/16 inches	1.0	Black on white	87
			Black on yellow	93
			White on black	95
STOP	6 by 1 inches	.8	Black on white	93
			Black on yellow	99
			White on black	97
CURVE	6 by ⅞ inches	.7	Black on white	84
			Black on yellow	88
			White on black	87
ZONE	5 by ⅝ inches	.6	Black on white	76
			Black on yellow	83
			White on black	91
SLOW	8 by 1 3/16 inches	.6	Black on white	84
			Black on yellow	87
			White on black	98

500-FOOT STATION

Sign	Size of letter	Time interval	Color combination	Percentage of correct readings
SLOW	8 by 1 3/16 inches	1.0	Black on white	42
			Black on yellow	58
			White on black	37
STOP	6 by 1 inches	.8	Black on white	30
			Black on yellow	47
			White on black	30
CURVE	6 by ⅞ inches	.7	Black on white	35
			Black on yellow	39
			White on black	37
ZONE	5 by ⅝ inches	.6	Black on white	33
			Black on yellow	44
			White on black	37
SLOW	8 by 1 3/16 inches	.6	Black on white	34
			Black on yellow	52
			White on black	37

At the maximum distance, 500 feet, at which station 11 observers made 600 observations, the following data were obtained. On the large SLOW sign at 1.0 second, 42 percent correct for black on white, 58 percent for black on yellow, and 37 percent for white on black was registered. At 0.8 second, with the STOP sign having 6-inch letters, the black on white shows 30 percent, black on yellow 47 percent, and white on black 30 percent correct. At 0.7 second with a 6-inch letter having a narrow stroke (⅞-inch) the correct readings were: Black on white 35 percent, black on yellow 39 percent, and white on black 37 percent correct. The increase in percentage of correct readings for the black on white and white on black combinations is probably due to the fact that the large CURVE symbol (arrow) was displayed on the CURVE sign, thus introducing an element of suggestion to the observer. Changes on this sign consisted of substituting large letters R or L for the arrow or TURN for CURVE. At 0.6 second the ZONE sign was first used and the changes or transpositions consisted of interchanging the words HOSPITAL and SCHOOL. These letters are 5 inches in height and correct registration was: Black on white 33 percent, black on yellow 44 percent, and white on black 37 percent.

It is interesting to note that at this point several observers were asked if the copy was clearly read or if the identification was made from the picture as outlined

by the length of the words. A number of observers claimed that they actually read the words but perhaps a greater number identified the sign by reading the shorter word and guessed at the longer. Comparing the readings of the ZONE sign at 0.6 second against the 8-inch letter of the SLOW sign at the same interval, we find an increase of 1 percent in efficiency of the larger letter over the smaller in the case of black on white and 8 percent in the case of the black on yellow, while the white on black registered no difference.

RESULTS OBTAINED BY OTHER INVESTIGATORS DISCUSSED

As previously indicated, exhaustive investigations of sign color combinations have been made in university and commercial research laboratories and by State highway departments, but apparently this is the first time that there has been an opportunity for estimating personal percipiency or reaction of the road user as measured by a definitely controlled time interval or factor. In effect, the percentages quoted confirm deductions made in some of the laboratory tests although these may have dealt primarily with the visibility of color used on automobile tags and not exclusively with highway signs.

From the investigations made last year by Prof. A. R. Lauer, of the department of psychology, Iowa State College, under the auspices of the National Research Council, it appears that yellow for backgrounds is better than orange; but Professor Lauer's deductions that light backgrounds are always better than dark was not completely verified. In connection with the studies made in the office of the New Jersey Traffic Commission at Trenton, Mr. A. H. Vey reports that it appears that white on a black background gave from 15 to 20 percent greater distance legibility. No such superiority of white on black over black on white is indicated in these tests. Mr. Vey further expresses the opinion, however, that black on white is more adaptable under certain conditions, which will depend upon the background; and will probably give better service. He suggests that any objections to the black background might be overcome to a large extent by designing a wide white border around the sign.

In his treatise on Color and Its Application, 1921, Mr. M. Luckiesh, quoting a report from Le Courrier du Livre, gives the combination black on yellow as first and white on black as tenth in his list of 13 color combinations. The customary black on white is sixth. This rating is borne out by these investigations only as applying to the black on yellow. On the other hand, in an unpublished investigation of automobile license tags made by the National Bureau of Standards, it developed that the black on yellow combination was fourth in the list of 25 combinations investigated. Black on white was eighth and white on black was eleventh. These three combinations gave 93.2, 91.1, and 87.7 percent, respectively, of correct readings.

These ratings, while supporting in a measure the present investigations, are not necessarily conclusive in that the time interval of free sight was not considered, nor was any attempt made to relate the observing time to the distance at which the tags were legible.

RAPIDITY OF RECOGNITION OF SHAPE, COLOR, AND LEGEND

The rapidity with which the average individual observer could select or pick out a sign from its surrounding background and recognize its shape, color, or legend was tested at time intervals of 0.5, 0.4, and 0.3 second. In only a negligible number of instances and at the minimum distance were observers able to identify the sign by any of these characteristics at intervals shorter than 0.3 second. The SLOW sign with an 8-inch letter and the STOP sign with a 6-inch letter were used for this purpose. In all, 600 observations were made by 50 observers. The average percentages of correct readings, combining all three factors and including all distances, are given in table 4.

Here the superiority of the black on yellow is again apparent, except at the 0.5-second time period, where black on white has a 3-percent advantage. This superiority is most pronounced at the minimum of 0.3 second.

TABLE 4.—*Percentage of correct readings of shape, color and legend combined, at short time intervals; average for all distances*

Sign	Size of letter	Time interval	Color combination	Percentage of correct readings
	Inches	*Seconds*		
STOP	6	0.5	Black on white	78
			Black on yellow	75
			White on black	60
SLOW	8	.4	Black on white	54
			Black on yellow	64
			White on black	56
STOP	6	.3	Black on white	28
			Black on yellow	45
			White on black	33

In tables 5, 6, and 7, the component characteristics of shape, color, and legend, are separated. Table 5 gives the data for recognition by the shape of the sign.

TABLE 5.—*Correct recognition of shape at short time intervals; average for all distances*

Sign	Shape of plaque	Time interval	Color combination	Percentage of correct readings
		Seconds		
STOP	Octagon	0.5	Black on white	85
			Black on yellow	77
			White on black	59
SLOW	Diamond	.4	Black on white	61
			Black on yellow	66
			White on black	55
STOP	Octagon	.3	Black on white	32
			Black on yellow	49
			White on black	33

The advantage of the black on white at 0.5 second may possibly be attributed to the contrasting background aided by a bright afternoon sunlight falling directly on the sign. This advantage, however, did not obtain at the shorter intervals, at which black on yellow retains its superiority.

For recognition of color, a feature which is perhaps the most important of the three factors, in that the element of color contrast with surrounding backgrounds is paramount, the results were as shown in table 6.

These percentages clearly indicate the superiority of the Federal yellow for ground color. This superiority is most marked at the shortest interval. The high percentage of the black on white at 0.5 second seems to check the assumption previously made that bright sunlight materially assisted in accentuating the contrast with the surrounding background.

Table 7 gives the results for correct readings of the sign legend only.

TABLE 6.—*Correct recognition of color at short time intervals: average for all distances*

Sign	Time interval	Color combination	Percentage of correct reading
	Seconds		
STOP	0.5	Black on white	
		Black on yellow	
		White on black	
SLOW	.4	Black on white	
		Black on yellow	
		White on black	
STOP	.3	Black on white	
		Black on yellow	
		White on black	

TABLE 7. *Correct reading of legend at short-time intervals: average for all distances*

Sign	Size of letter	Time interval	Color combination	Percentage of correct reading
	Inches	*Seconds*		
STOP	6	0.5	Black on white	
			Black on yellow	
			White on black	
SLOW	8	.4	Black on white	
			Black on yellow	
			White on black	
STOP	6	.3	Black on white	
			Black on yellow	
			White on black	

At these intervals no attempt was made to check the observer's accuracy by transposing or otherwise changing the letters and the data may not be conclusive because it was felt that at least some observers had subconsciously memorized previous indentifications. As the interval shortens the decreased accuracy is consistent with the decreases in the shape and color readings. The general averages for the three short-time intervals and at all the distances but for each of the elements of recognition, as shown in table 8, gives a very concrete idea of the weight the respective components bear to complete recognition.

TABLE 8.—*General averages of correct readings of shape, color, and legend at 0.5, 0.4, and 0.3 second for all distances*

Color combination	Shape	Color	Legend
	Percent	*Percent*	*Percent*
Black on white	59	63	36
Black on yellow	64	77	41
White on black	49	60	29

Here the fact that black on white is superior to white on black in the first and second cases is possibly due to the element of glare which did not affect recognition of either shape or color. For legibility of legend, however, white on black has 3 percent advantage over black on white, and black on yellow is more readily picked up than either of the other combinations.

Reading at the mid or critical distance of 350 feet before referred to and distinguishing between the factors shape, color, and legend, we have the following results at these short intervals. For recognition of shape, black on white gave 51 percent, black on yellow 55 percent, and white on black 42 percent. Here, as in the general average, the superiority of the white over the black background is again demonstrated, but the lead of the yellow is maintained. For recognition of color, black on white gave 55 percent; black on yellow 73 percent; and white on black 64 percent. This reverses the black and white order as shown by the previous percentages. For recognition of legend, black on white gave 37 percent; black on yellow 50 percent; and white on black 42 percent, again confirming the general averages.

At the minimum distance (200 feet) and for the same time intervals, that is, 0.5, 0.4, and 0.3 second, the following results were obtained. For recognition of shape, black on white registered 55 percent correct, black on yellow 81 percent, and white on black 85 percent. As to color, black on white registered 65 percent, black on yellow 89 percent, and white on black 85 percent. As to legend, black on white gave 58 percent, black on yellow 77 percent, and white on black 87 percent correct. At this short range, it is difficult to understand why the white on black sign should be so distinctly superior to the black on white. For color, however, the black on yellow is still superior to the other combinations.

At the maximum distance of 500 feet there is a reversal of the relative ratings of the black and white combinations, as compared with those obtained at the minimum distance of 200 feet, as the following will show. For shape, black on white registered 65 percent, black on yellow 65 percent, and white on black 23 percent correct. For color, black on white registered 65 percent, black on yellow 81 percent, and white on black 43 percent. For legend, black on white registered 17 percent, black on yellow 14 percent, and white on black 10 percent. For all three characteristics, black on white is superior to white on black, but black on yellow still leads. Especially significant is the high percentage of correct color readings, 81 percent for black on yellow as against 65 and 43 percent for the black and white combinations. With the varied background of foliage and artificial culture, it was not difficult to recognize the yellow even though the time interval was reduced to 0.3 second. The white sign contrasted distinctly with the natural background.

Analysis of the recognition of each of the three component elements by time interval at the critical station (350 feet) clearly presents the comparison sought and it is unnecessary to break the data down into the time factors obtained at the other stations. The consistency of the data obtained at the 350-foot station with the comparisons shown in the general averages for all distances is significant. The detailed data for this station are given in table 9.

REASONS FOR SUPERIORITY OF BLACK ON YELLOW DISCUSSED

The foregoing data are sufficiently exhaustive to indicate definitely that the black legend on the yellow background has a distinct advantage over either the white on black or black on white combinations and that the selection of this combination by the American Association of State Highway Officials is fundamentally correct.

In arriving at this conclusion two features which enhance its value as a signal, the rapidity with which a driver's attention is attracted to the yellow sign and the conspicuousness of its contrast with surrounding backgrounds, have been duly weighed. By signal value is meant the maximum distance at which a sign can be identified as a sign regardless of the distance at which its message becomes clearly readable. At distances of 350 feet or greater, with the 0.5 to 0.3 second

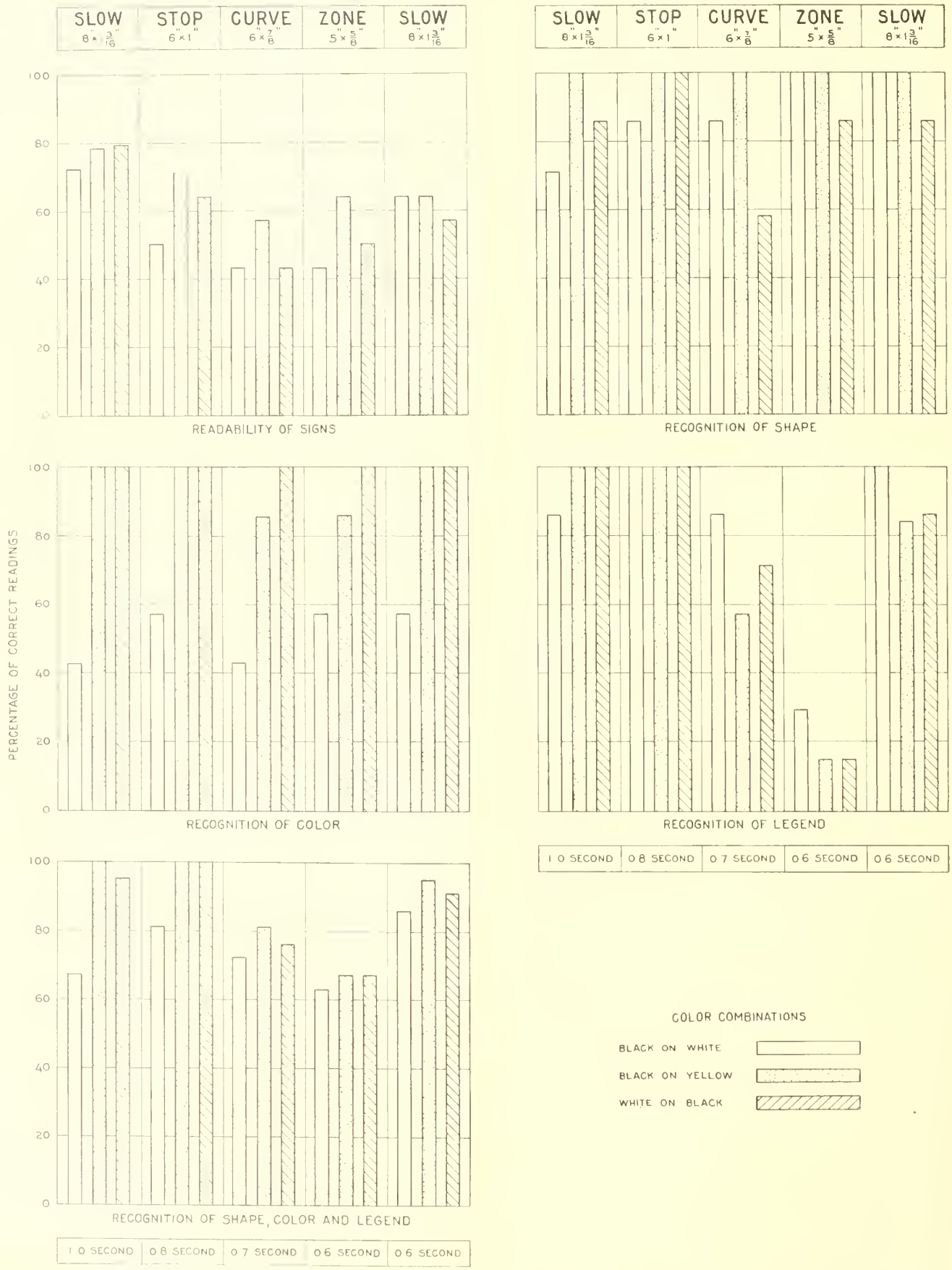

FIGURE 8.—NIGHT OBSERVATIONS ON NONLUMINOUS SIGNS: COMPARATIVE READABILITY AND RECOGNITION OF SHAPE, COLOR, AND LEGEND.

TABLE 9.—*Correct recognition of shape, color, and legend at 0.5, 0.4, and 0.3-second intervals at 350-foot station*

Time interval	Recognition of—	Color combination	Percentage of correct readings
Seconds			
0.5	Shape	Black on white	74
		Black on yellow	68
		White on black	58
.5	Color	Black on white	80
		Black on yellow	80
		White on black	71
.5	Legend	Black on white	55
		Black on yellow	61
		White on black	45
.4	Shape	Black on white	60
		Black on yellow	51
		White on black	39
.4	Color	Black on white	60
		Black on yellow	80
		White on black	80
.4	Legend	Black on white	41
		Black on yellow	51
		White on black	58
.3	Shape	Black on white	19
		Black on yellow	50
		White on black	29
.3	Color	Black on white	25
		Black on yellow	56
		White on black	35
.3	Legend	Black on white	13
		Black on yellow	38
		White on black	23

time intervals, particular advantage is seen for the yellow background and at 300 feet or less the wording becomes progressively more legible. At 40 miles per hour a car will cover 59 feet per second, so that if the driver's attention is attracted to the sign at 350 feet, he will have 3 seconds in which to arrive at a point approximately 180 feet in front of the sign, where legibility is reasonably assured.

NIGHT OBSERVATIONS ON DAYLIGHT SIGNS

As a proper supplement to the comparisons made in daylight, 260 observations of signs unequipped with reflecting units were made at night. The observing personnel consisted of 4 females and 6 males, a total of 10 observers. The general location was the same as that used in the daylight tests except that on account of side illumination from street lamps and adjacent buildings it was necessary to change the position of the test object. Automobile headlights of a type approved by the Eastern Conference of Motor Vehicle Administrators were mounted in front of the observer's screen previously described. The observing platform was set up in the right hand driving track of the highway and the signs were placed on the road shoulder approximately 2 feet to the right of the pavement edge. The headlight beams were directed straight ahead with the vertical angle such that the cut-off of the beam fell approximately on the upper third of the sign, thus simulating as far as possible actual driving conditions (fig. 3).

By experiment it was found useless at night to attempt to read the daylight signs at distances greater than 200 feet. For this reason the data presented are confined to that distance. At distances in excess of 200 feet, the black sign could not be distinguished from the surrounding background, except when piles of snow furnished a contrasting field. In a lesser degree this is also true of the yellow background. The white background seemed to merge into the illuminated pavement or snowbank; and if it was distinguished at all the observers invariably claimed to have seen a yellow sign. This may have been due to selective reflection or to the yellow hues of the light beam. Briefly, it was felt that the data obtained at distances greater than 200 feet would not be susceptible of any real differentiation and could not serve as a basis for comparison. At 200 feet the black sign was in distinct contrast with the snow background, resulting in 100 percent correct recognition of the color, as against 74 percent for legend and 83 percent for shape.

As in the daylight tests, four signs were used, i.e., SLOW, STOP, CURVE, and ZONE. The time intervals for reading the shape, color, and legend were 1.0, 0.8, 0.7, 0.6, and 0.5 second.

In order to determine the general readability of the signs under these conditions the device was employed of transposing letters and determining whether the transpositions could be picked up by the observer. The data on comparative readability are given in the first panel of figure 8. Average values for the different color combinations were as follows:

	Percent correct
Black on white	54
Black on yellow	67
White on black	59

The remaining panels of figure 8 give the data on recognition of shape, color, and legend, respectively, and of shape, color and legend combined. Average values are given in table 10.

TABLE 10.—*Night observations on daylight signs; average percentages of correct readings of shape, color, and legend for all time periods at the 200-foot station*

Recognition of—	Color combination	Percentage of correct readings
Shape	Black on white	88
	Black on yellow	100
	White on black	83
Color	Black on white	51
	Black on yellow	93
	White on black	100
Legend	Black on white	89
	Black on yellow	71
	White on black	74

Thus it seems that as a sign the yellow background contrasted more distinctly with its surroundings and was more easily recognized than either the black on white or white on black; but the legend was more legible on the latter two signs. It should be remembered, however, that the yellow tinge of the light beam threw a distinct yellowish hue on the white sign. The probability of this effect was called to the attention of the observers and it is possible that they were influenced thereby to a greater or less degree in their decision, calling the white sign yellow.

NIGHT OBSERVATIONS ON SIGNS EQUIPPED WITH REFLECTING UNITS

In carrying on the night tests for signs equipped with reflecting units (buttons) the same general procedure as for the daylight signs was followed but, inasmuch as a differentiation between reflectors of different diameters was sought, it was necessary to make up three sets of letters required for each sign. As the painted bristol board letters used on the daylight signs could not be perforated to receive the reflectors, letters punched to receive the reflectors were stamped out on aluminum sheets. These letter silhouettes were countersunk into wood blocks which in turn fitted into their respective places across the body of the sign proper. This

permitted rapid change or transposition of letters equipped with different-sized buttons. The buttons used were white or colorless and of three diameters, viz, 0.95-inch, 0.76-inch, and 0.58-inch. These are the overall dimensions of the mounting flange. The optical diameters were 0.76, 0.58, and 0.41 inch, respectively. Figure 9 shows examples of the signs with reflector buttons and the manner in which they were fabricated.

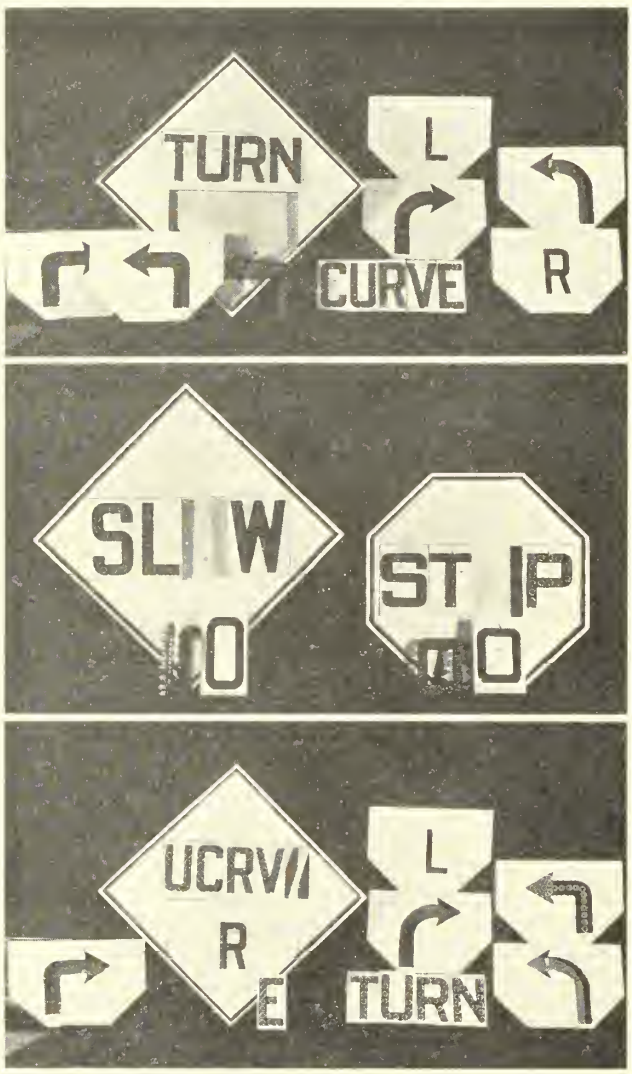

FIGURE 9.—EXAMPLES OF SIGNS EQUIPPED WITH REFLECTOR BUTTONS, SHOWING METHOD OF FABRICATION.

Because of the different shapes and superficial areas of the various letters, spacing between reflecting elements could not be made constant except at the expense of proper definition. The letters L, T, and E, as shown in table 11, serve as a basis for the comparison of the spacing of the different-sized buttons and the relative areas occupied by them.

The SLOW and STOP signs, and CURVE and TURN signs with directional arrow symbols were used. The SCHOOL and HOSPITAL ZONE signs were not included. Signs of this character are not usually equipped with reflecting buttons; the width of the letter stroke would admit the 0.58-inch button only and even this would leave practically no margin.

There were 13 observers, 11 male and 2 female. The total number of observations was 652. Observing stations included the 200 and 300-foot stations for

TABLE 11.—*Proportions of areas of letters occupied by reflecti buttons of different sizes and spacing*

Letter	Size	Area	Diameter of button	Number of buttons	Spacing, center to center	Area occupied by buttons	Per centa of tot area
		Sq. ins.	*Inches*		*Inches*	*Sq. ins.*	
L	8 by 1 3/16 inches	12.84	0.95	8	1 3/8	5.67	
T	6 by 1 inches	9.31	.95	7	1 1/4	4.96	
E	6 by 7/8 inches	10.06	.95	(1)	(1)	(1)	(1)
L	8 by 1 3/16 inches	12.84	.76	11	1	5.09	
T	6 by 1 inches	9.31	.76	11	7/8	5.09	
E	6 by 7/8 inches	10.06	.76	12	7/8	5.55	
L	8 by 1 3/16 inches	12.84	.58	14	3/4	3.70	
T	6 by 1 inches	9.31	.58	11	1 3/16	2.91	
E	6 by 7/8 inches	10.06	.58	16	3/4	4.23	

[1] The letter E in this series did not admit the large button.

complete data and the 350-foot station for confirma tory data on the 0.95-inch button only.

In the night observations on signs equipped wit buttons the ground color was not considered especiall important, and was not used as a factor.

The general average percentages of correct reading for the four signs used in these observations are show in figure 10. At 200 feet, the general averages of co

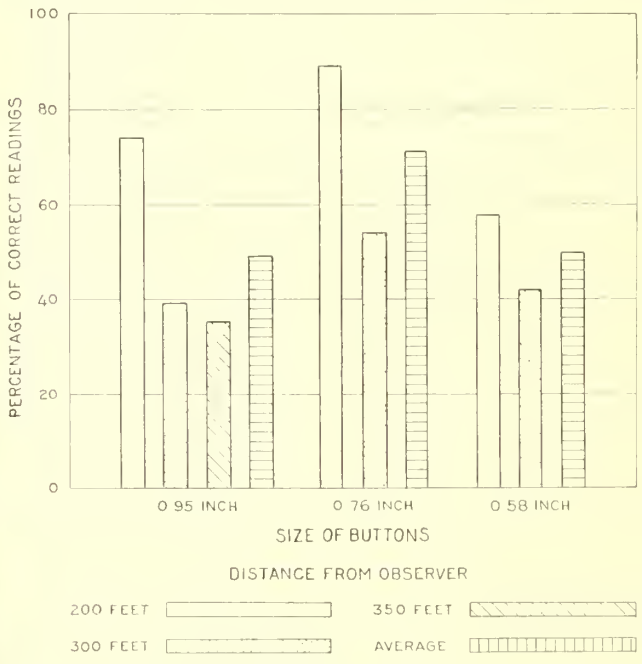

FIGURE 10.—NIGHT OBSERVATIONS ON LUMINOUS SIGN AVERAGE PERCENTAGES OF CORRECT READINGS FOR DI FERENT SIZES OF BUTTON.

rect readings of the four signs and for all time period are: On the 0.95-inch button 74 percent correct; o the 0.76-inch button 88 percent correct; and on th 0.58-inch button 58 percent correct.

At 300 feet, the results are as follows: On the 0.9 inch button 39 percent correct; on the 0.76-inch butto 54 percent correct; and on the 0.58-inch button 42 pe cent correct.

At these distances the 0.76-inch button is apparentl the most efficient. Several observers intimated that th reflection from the 0.58-inch button dispersed in such manner as to make the word illegible. On the othe hand the small button was noted to be particularl brilliant, and thus a great measure of signal value c attention arresting attraction may attach to its us

The large 0.95-inch button has an advantage over the smallest button at 200 feet but at 300 feet the condition is reversed and probably can be attributed to the fact that at the greater distance glare on the 0.58-inch button proportionately decreased. Observers also intimated that the 0.95-inch button lacked in brilliancy when compared with the other two.

Figure 11 gives the detailed data for each time period and size of letter. Table 12 gives the same material in tabular form for the 200- and 300-foot stations.

TABLE 12.—*Night observations on luminous signs; average percentage of correct readings by time period with different sizes of reflector button*

AT 200 FEET

Sign	Size of letter	Time interval	Percentage of correct readings		
			0.95-inch buttons	0.76-inch button	0.58-inch button
		Seconds			
SLOW	8 by $1\frac{3}{16}$ inches	1.0	81	94	69
STOP	6 by 1 inches	.8	81	94	50
TURN	6 by 1 inches	.7	81	88	62
CURVE	6 by $\frac{7}{8}$ inches	.6	—	88	44
STOP	6 by 1 inches	.6	69	81	62
SLOW	8 by $1\frac{3}{16}$ inches	.5	56	88	62

AT 300 FEET

SLOW	8 by $1\frac{3}{16}$ inches	1.0	60	85	75
STOP	6 by 1 inches	.8	30	60	60
TURN	6 by 1 inches	.7	35	55	35
CURVE	6 by $\frac{7}{8}$ inches	.6	—	45	20
STOP	6 by 1 inches	.6	35	45	40
SLOW	8 by $1\frac{3}{16}$ inches	.5	35	35	40

The two readings at the 0.6-second period were taken to check accuracy. The stroke of letters in CURVE is narrower than in STOP and the word itself has a more crowded appearance and is more difficult to read unless the meaning is suggested by using the direction arrow. As in the case of the TURN sign, the letters R or L were frequently substituted for the direction arrow as an additional check. The width of stroke of letters in the CURVE sign is too narrow to admit the large button.

These tabulations confirm deductions made from the general averages but it is especially interesting to note the apparent efficiency of the 0.76-inch button at 200 feet. With the 8-inch letter of the SLOW sign at 1.0 second, the readings were 94 percent accurate and at 0.5 second on the same sign this had decreased by 6 percent. In the case of the 0.95-inch button the decrease was 25 percent. With the 0.58-inch button there was an increase of 18 percent between the readings of the congested lettering in the CURVE sign at 0.6 second and the more open lettering on the STOP sign at the same interval. This wide difference also occurs at the 300-foot station and apparently justifies the assumption that legibility is greatly impaired by glare. The pattern on the word CURVE contributes largely to this feature.

At the 350-foot station the average of correct observations of the 0.95-inch button fell off to 35 percent and it was not considered worth while to increase the number of observations or to go to greater distances. Several signs, however, were carried back to 450, 500, and 600 feet to ascertain signal value, disregarding time period for readability of the legend. The distinct brightness of the 0.58-inch button was again noted to a degree not obtained with either of the larger-sized buttons.

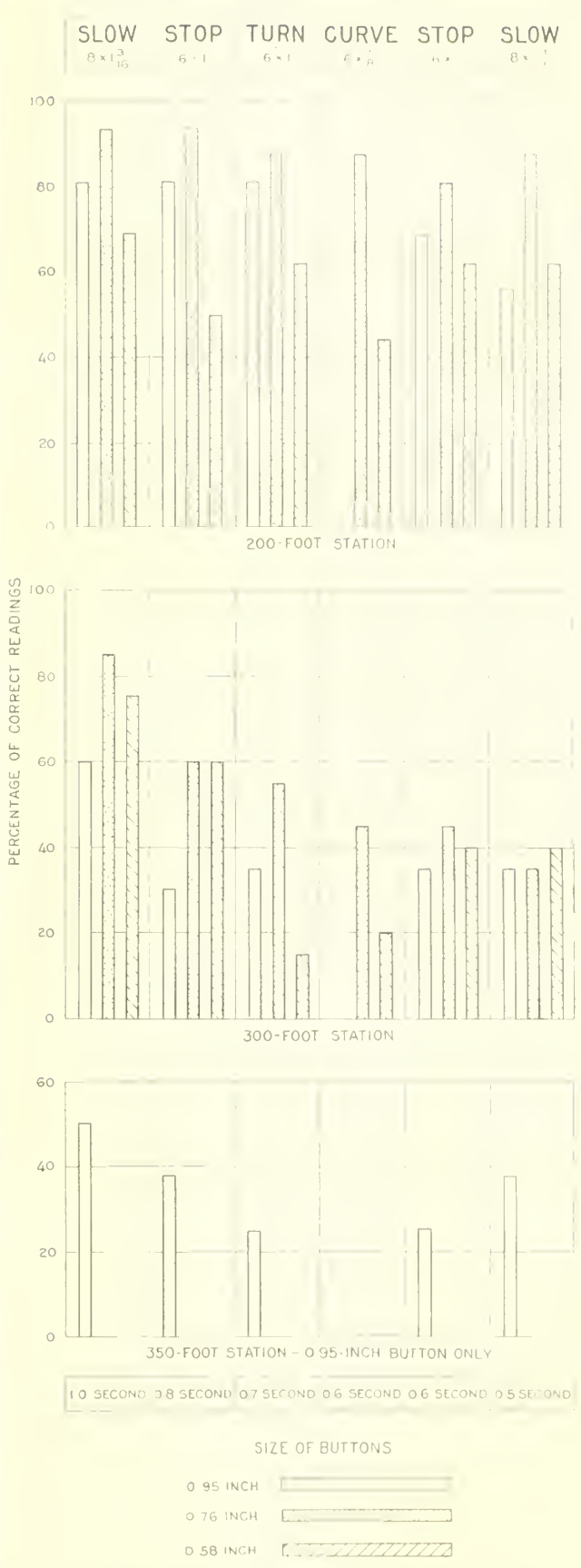

FIGURE 11.—NIGHT OBSERVATIONS ON LUMINOUS SIGNS. DETAILED PERCENTAGES OF CORRECT READINGS FOR DIFFERENT SIGNS, TIME INTERVALS, AND SIZES OF BUTTON.

LONG-RANGE IDENTIFICATION ASSISTED BY OUTLINING MARGIN OF SIGN WITH REFLECTOR BUTTONS

At these distances observations were also made of signs outlined by inserting 0.95-inch buttons in the margin as shown in figure 12. The shapes thus out-

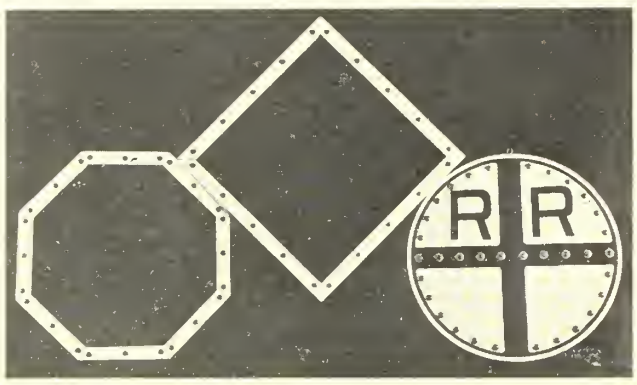

FIGURE 12.—SIGNS WITH MARGINS OUTLINED BY REFLECTOR BUTTONS.

lined were distinctly visible and could readily be described at all distances up to 600 feet or more. At extreme distances, however, some observers confused the octagon with the circular railroad sign. The characteristic difference in the illuminated outlines of these signs is the luminous cross bar carried on the railroad disc. Apparently this feature was noted by some observers and to that extent assisted in accurate differentiation. To other and presumably less acute observers the luminous outlines presented no material difference at distances greater than 500 feet. For long-range identification of a warning sign at night, the definition of shape by marginal installations of reflecting buttons has much to recommend it.

The idea of attributing special significance to the shape of the sign plaques was one of the first considerations in the design of standard highway signs adopted by the American Association of State Highway Officials. Obviously rapid and perhaps subconscious recognition of the sign warning by this method is largely a matter of education of the driving public. It is believed, however, that the indication of potential hazards existing on the highway as shown by a series of differently shaped signs will more readily appeal to the average road user than will a series of symbols such as has been recommended in some of the codes adopted by foreign governments and suggested for international usage. Outlining the shape of the sign by reflecting buttons for night use undoubtedly merits further investigation than the limited scope afforded by these tests.

RURAL AND CITY DRIVING CONDITIONS COMPARED

The night observations described up to this point were taken under full headlight illumination which at 200 feet was 0.2 foot-candle on the sign and at 300 feet was 0.1 foot-candle. Every effort was made to obtain actual rural road driving conditions. Surrounding street lights were extinguished and window lights from adjacent buildings were screened.

The investigation would not have been complete without an attempt to compare rural with city driving and to this end lights in adjacent buildings and street lights were turned on, and the headlights were dimmed. Observations, however, were restricted to the 300-foot station and four observers made 136 observations with the following results. Readings on the 0.95-inch button were 35 percent correct, on the 0.76-inch button 63 percent correct, and on the 0.58-inch button 81 percent correct. In figure 13 these results are compared with those obtained at the 300-foot station under rural driving conditions (full headlights).

Attention is particularly called to the increased efficiency of the small button under the city driving conditions. An appreciable increase is also found in the 0.76-inch button. This is probably due to the absence of glare under subdued lights or to the influence of the extraneous lighting combining with the headlight beam.

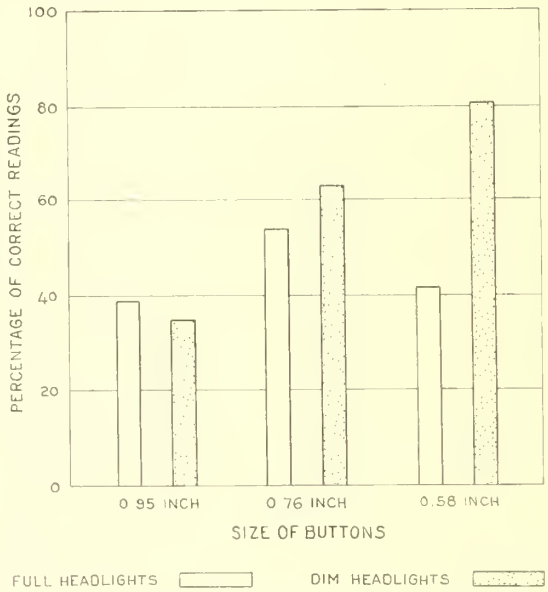

FIGURE 13.—NIGHT OBSERVATIONS ON LUMINOUS SIGNS: COMPARATIVE READABILITY OF SIGNS UNDER CITY AND RURAL DRIVING CONDITIONS AT 300-FOOT STATION.

DAYLIGHT OBSERVATIONS OF SIGNS EQUIPPED WITH REFLECTING BUTTONS

Responsible manufacturers have laid great stress upon the reflecting values of their individual products and arguments in favor of this or that design have invariably emphasized advantages of brilliancy of the reflected beam, breadth of the angle of reflection, focal length, and other special characteristics of design, but have not considered their effect on the daylight legibility of the sign wording. It is recognized that many of the hazards inherent in a roadway, such as abrupt curvature, road intersections, railroad grade crossings, etc., readily seen in daylight, require adequate sign protection at night. This condition, however, should not operate to minimize the daylight values of the warning signs. The tests on signs equipped with buttons were, therefore, extended to cover daytime observations of the SLOW, STOP, TURN, and CURVE signs at distances from 200 to 400 feet. Black letters on yellow background and white letters on black baekground only were used.

Twenty-nine observers, of whom 9 were female and 20 male, made a total of 2,098 observations.

Figure 14 gives the percentages of correct readings in daylight, at distances from 200 to 400 feet, for signs equipped with the three sizes of button. For purposes of comparison, the average percentages obtained at these distanees with daylight signs having the same backgrounds (black on yellow and white on black) are also plotted in this figure. Table 13 gives the com-

parative data in tabular form. Detailed data for the various distances are given in figure 15.

TABLE 13.—*Comparative readability of luminous and nonluminous signs under daylight conditions; average percentages of correct readings at each station for both black on yellow and white on black*

	Percentage of correct readings			
	At 200 feet	At 300 feet	At 350 feet	At 400 feet
Daylight signs	92	65	70	43
0.95-inch buttons	55	68	31	22
0.76-inch buttons	74	63	25	30
0.58-inch buttons	64	67	27	25

The percentages for the daylight signs were obtained from observations made in relatively bright sunlight during the summer months, whereas observations for the button-equipped signs were made in January with the sky overcast during a portion of the time. At the 350-foot station, observations were made during heavy snowfall, which probably accounts for the abrupt falling off in the percentages computed for that station. At the initial station (200 feet), however, atmospheric conditions affecting visibility were the same at both periods of observation and it is interesting to note that with approximately 49 percent of the area of the black letters occupied by the buttons, the legibility materially decreased. A tabulation of the detailed data for the 200-foot station is given in table 14.

TABLE 14.—*Comparative readability of luminous and nonluminous signs under daylight conditons at 200 feet*

Sign	Time interval	Percentage of correct readings			
		Without buttons	With buttons		
			0.95-inch	0.76-inch	0.58-inch
	Seconds				
SLOW	1.0	94	81	94	94
STOP	.8	98	38	87	81
TURN	.7	87	68	56	62
CURVE	.6	87	.....	50	38
STOP	.6	92	44	75	56

Increase in legibility of the STOP sign at 0.6 second over the CURVE sign for the same period may be attributed to the wider stroke and more open distribution of the letters in the STOP sign.

Table 15 gives the comparative data for the 300-foot station.

TABLE 15.—*Comparative readability of luminous and nonluminous signs under daylight conditions at 300 feet*

Sign	Time interval	Percentage of correct readings			
		Without buttons	With buttons		
			0.95-inch	0.76-inch	0.58-inch
	Seconds				
SLOW	1.0	81	75	86	93
STOP	.8	76	75	67	82
TURN	.7	59	67	41	47
CURVE	.6	44	.....	46	47
STOP	.6	66	57	71	71

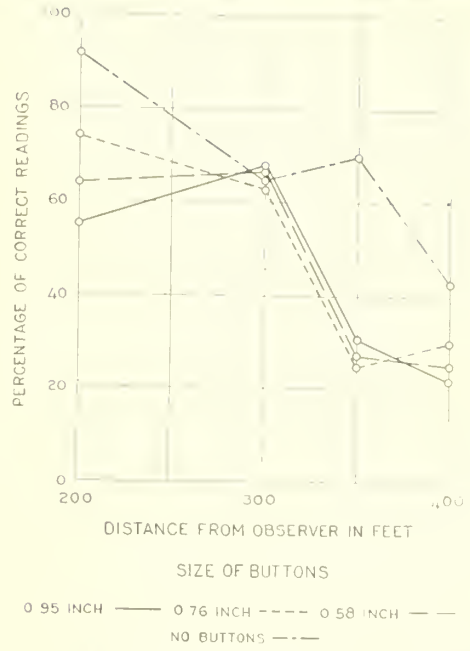

FIGURE 14.—COMPARATIVE READABILITY OF LUMINOUS AND NONLUMINOUS SIGNS UNDER DAYLIGHT CONDITIONS; AVERAGE PERCENTAGES OF CORRECT READINGS AT EACH STATION FOR BOTH BLACK ON YELLOW AND WHITE ON BLACK.

The visibility of the button-equipped signs at this station was considerably lessened by overcast or cloudy sky but the consistency of the results with the comparisons at 200 feet (table 14) should be noted.

The data for the 350- and 400-foot stations are given in table 16. At the 350-foot station observations of the button-equipped signs were made during a severe snow storm. Comparison, therefore, with the results obtained during bright sunlight from daylight signs will not form a basis for definite conclusions. The observations made under the differing atmospheric conditions, however, are decidedly interesting.

TABLE 16.—*Comparative readability of luminous and nonluminous signs under daylight conditions at 350 and 400 feet*

Distance from observer	Sign	Time interval	Percentage of correct readings			
			Without buttons	With buttons		
				0.95-inch	0.76-inch	0.58-inch
Feet		*Seconds*				
350	SLOW	1.0	77	59	63	58
	STOP	.8	77	34	28	33
	TURN	.7	65	21	11	11
	CURVE	.6	63	.....	15	14
	STOP	.6	67	21	15	38
400	SLOW	1.0	59	53	64	64
	STOP	.8	39	16	25	27
	TURN	.7	42	8	3	3
	CURVE	.6	29	.....	30	14
	STOP	.6	41	8	22	16

The conditions of visibility at the 400-foot station were similar during the observations on both the button equipped and the unequipped signs; and the decreased efficiency, particularly for the 0.7-second period, can be attributed directly to the lack of contrast between the letters and sign background due to the buttons.

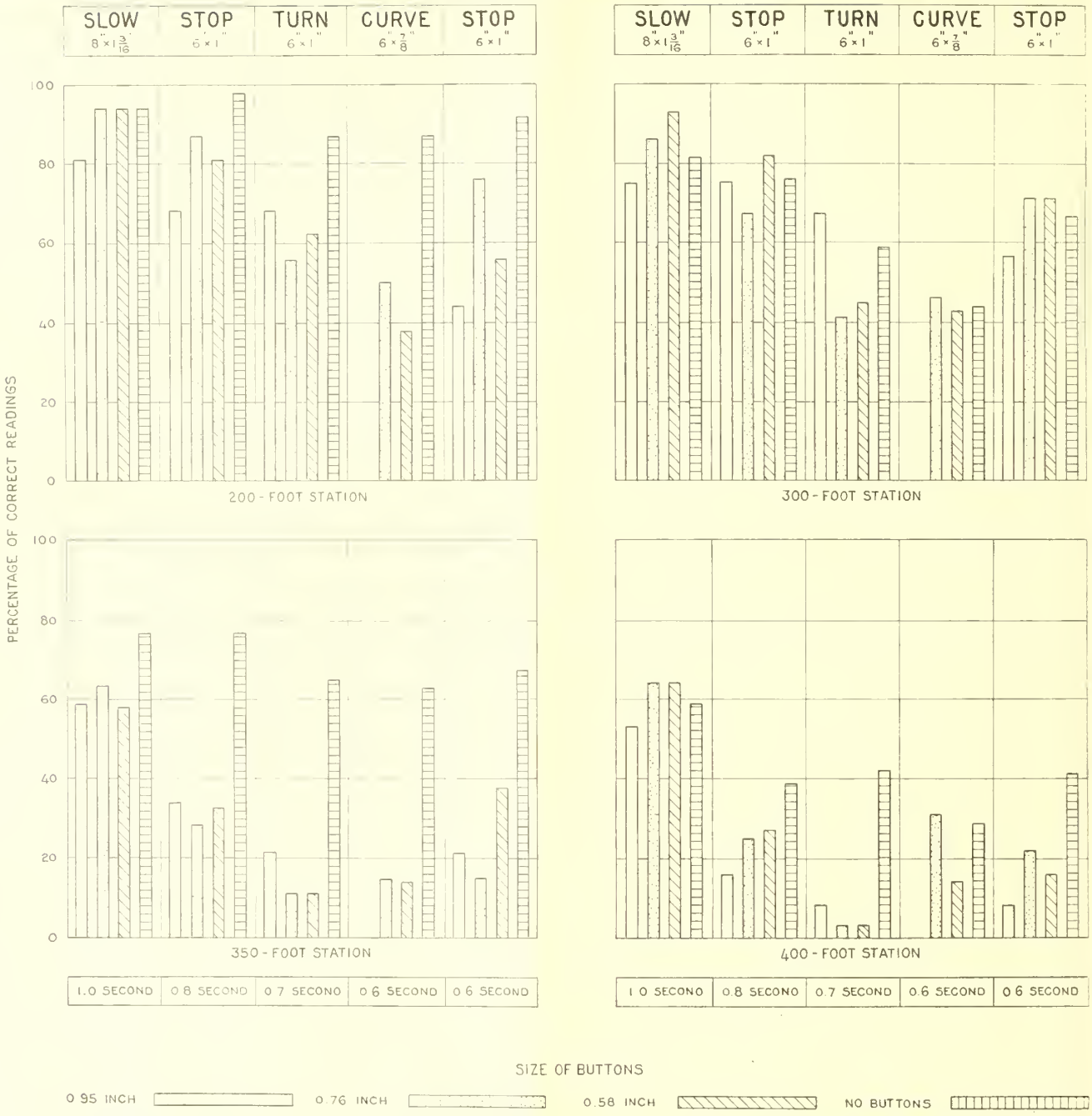

FIGURE 15.—COMPARATIVE READABILITY OF LUMINOUS AND NONLUMINOUS SIGNS UNDER DAYLIGHT CONDITIONS: DETAILED PERCENTAGES OF CORRECT READINGS FOR DIFFERENT SIGNS, TIME INTERVALS, AND SIZES OF BUTTON.

COMPARISON MADE OF RELATIVE LEGIBILITY OF BLACK AND WHITE LETTERS WHEN EQUIPPED WITH REFLECTING BUTTONS

Reference to table 11 in which are shown the percentages of the superficial areas of the letters L, T, and E occupied by buttons of different diameter, suggests the desirability of comparing the readability in daylight of black and white letters when equipped with reflector buttons. The figures and tables which follow give comparisons of percentages of correct readings, for legibility only, of signs with yellow background and black letters and those with black background and white letters. Figure 16 shows the variation of legibility with the distance from the observer. In the first panel of figure 17 the average values for each type of sign and each time interval used are given. The remaining panels of figure 17 give the detailed data for each station. Table 17 gives general averages for all distances, signs, and time periods.

TABLE 17.—*Comparative legibility of letters equipped with reflector*

Size of button	Color combination	Percentage of correct readings
Inches		
0.95	Black on yellow	4
	White on black	4
.76	Black on yellow	4
	White on black	4
.58	Black on yellow	4
	White on black	4

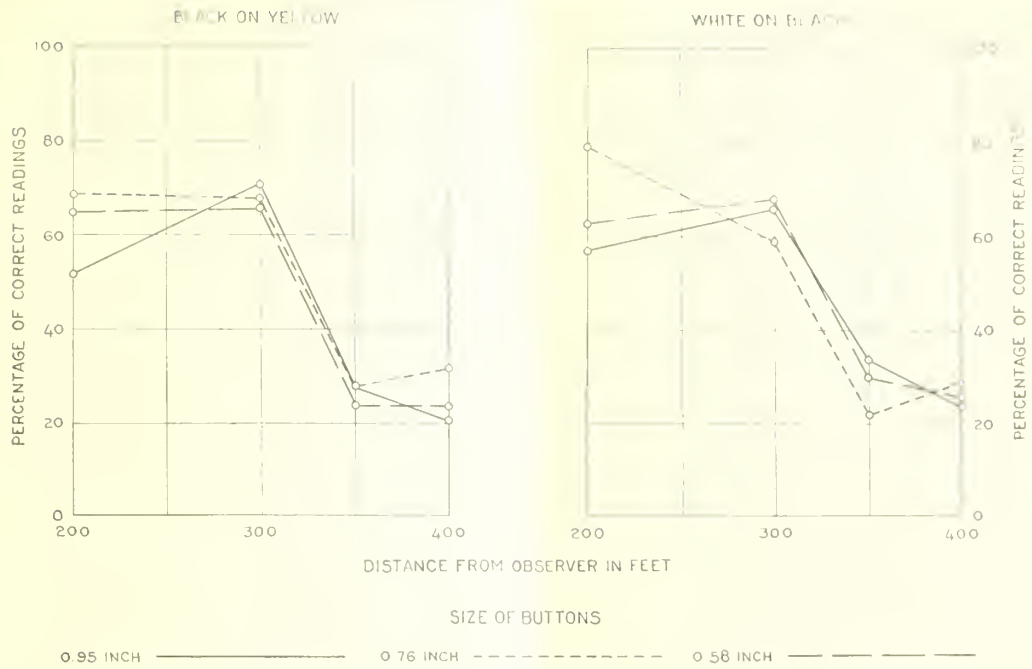

FIGURE 16.—COMPARATIVE READABILITY UNDER DAYLIGHT CONDITIONS OF BLACK AND WHITE LETTERS WHEN EQUIPPED WITH REFLECTOR BUTTONS OF VARYING DIAMETERS; VARIATION WITH DISTANCE FROM OBSERVER.

The differences in these averages are seen to be negligible and if we make the comparison station by station we obtain the extremely close check shown in table 18 and figure 16.

TABLE 18.—*Comparitive readability at various distances under daylight conditions of black and white letters equipped with reflecting buttons*

Distance from observer	Color combination	Percentage of correct readings		
		0.95-inch button	0.76-inch button	0.58-inch button
Feet				
200	Black letter	52	69	65
	White letter	57	79	63
300	Black letter	71	68	66
	White letter	66	59	68
350	Black letter	28	28	24
	White letter	34	22	30
400	Black letter	21	32	24
	White letter	24	29	26

Comparing this table also with the percentage readings for legibility of daylight signs given in figure 14, which indicated 92 percent correct at 200 feet, 66 percent at 300 feet, 70 percent at 350 feet, and 43 percent at 400 feet, we note that legibility has been greatly impaired by the introduction of the buttons if we disregard the unfavorable atmospheric conditions which obtained at the 350-foot station. At the 200-foot station the 0.95-inch button fell off from 92 percent to 52 percent; the 0.76-inch button to 69 percent, and the 0.58-inch button to 65 percent. These are for the black letters. At the 300-foot station there was a slight increase for the two larger buttons but no change for the 0.58-inch button is noted. At the 350-foot station, where snow fell during the observations of the button-equipped signs, the falling off for both the 0.95- and 0.76-inch button is from 70 to 28 percent, and from 70 to 24 percent for the 0.58-inch button. At the 400-foot station the decreases are from 43 to 21 percent for the 0.95-inch button; 43 to 32 percent for the 0.76-inch button, and 43 to 24 percent for the 0.58-inch button. The consistency of rise and fall or parallelism of the curves for the same button in the different colored letters is especially interesting.

The first panel of figure 17 is likewise interesting in this respect. It shows the average percentage of correct readings for legend only at all distances broken down into time periods for each of the three sizes of buttons in black and in white letters. Here the parallels are strongly emphasized. The CURVE sign was not used for the 0.95-inch button at the first 0.6-second period.

Finer analyses of these data at the 200-, 300-, 350-, and 400-foot stations are shown in the remaining panels of figure 17. In the case of the 0.76-inch buttons on black letters the STOP sign at 0.8 second shows a higher percentage than the SLOW sign at 1.0 second. This reversal may perhaps be attributed to the element of surprise in the initial reading in the case of two observers, the surprise having been overcome by the time the STOP sign was read. Between the CURVE sign at 0.6 second and the STOP sign at 0.6 second, the readings on this reflector were identical but with the large SLOW sign the white letter gave 100 percent while the black letter fell off to 63 percent. These results do not substantiate the hypothesis that a black letter is more impaired than a white letter by insertion of colorless buttons. There is some indication that such might be the fact at specific distances and for some of the time intervals but evidence that the white letter has a superior value when so equipped is not preponderant. It has been suggested that red buttons should be substituted for colorless buttons in the STOP sign on the ground that by so doing legibility of the word would suffer less impairment in daylight. Whatever the desirability of utilizing red buttons in this sign for emphasis may be, their use should not be based on the assumption of increased daylight visibility only, though red may have psychological potentialities as a mandate to stop.

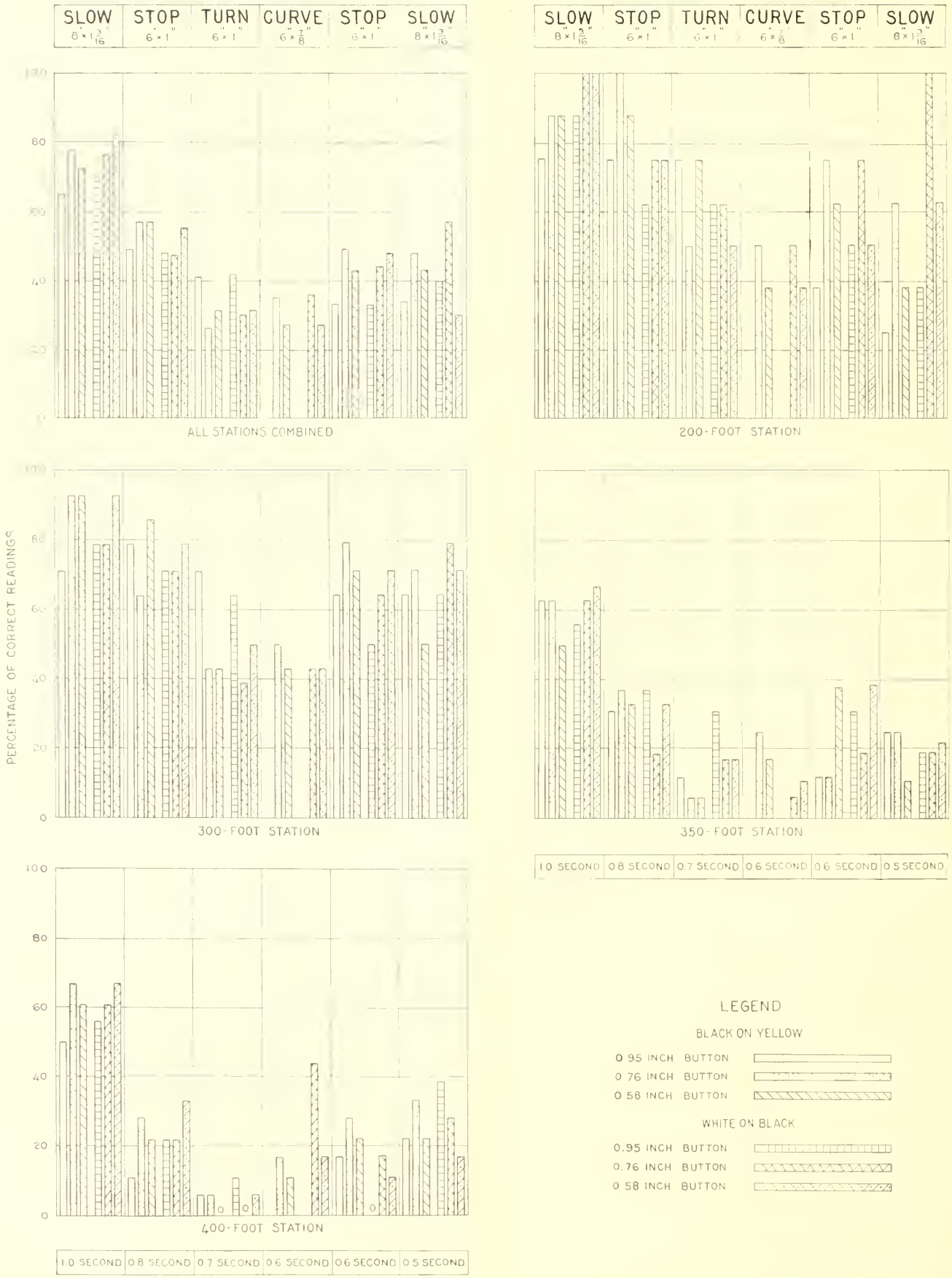

FIGURE 17.—COMPARATIVE READABILITY UNDER DAYLIGHT CONDITIONS OF BLACK AND WHITE LETTERS WHEN EQUIPPED WIT REFLECTOR BUTTONS OF VARYING DIAMETERS; DETAILED DATA FOR EACH SIGN, TIME INTERVAL, AND STATION.

TABLE 19.—*Number and spacing of reflector buttons used for letters of various standard sizes*

Size of buttons	Size of letters	Letter	Spacing of buttons	Number of button
		SLOW		
Inches			*Inches*	
0.95	8 by 13/16 inches	S	1 1/4	14
		L	1 1/8	8
		O	1 5/16	14
		W	1 3/8	16
				52
.76	do	S	1 1/8	17
		L	1	11
		O	1	17
		W	1	24
				68
.58	do	S	13/16	21
		L	3/4	14
		O	13/16	32
		W	13/16	32
				99
		STOP		
.95	6 by 1 inch	S	1 1/16	16
		T	1 1/4	7
		O	1 3/16	14
		P	1 3/16	12
				49
.76	do	S	15/16	18
		T	7/8	11
		O	7/8	18
		P	15/16	15
				62
.58	do	S	13/16	20
		T	13/16	11
		O	13/16	20
		P	13/16	17
				68
		CURVE		
.76	6 by 7/8 inch	C	7/8	12
		U	7/8	14
		R	7/8	17
		V	7/8	12
		E	7/8	12
				67
.58	do	C	3/4	11
		U	3/4	17
		R	3/4	19
		V	3/4	15
		E	3/4	16
				81
		TURN		
.95	6 by 1 inch	T	1 1/16	7
		U	1 1/16	11
		R	1 1/16	13
		N	1 1/16	13
				44
.76	do	T	1	9
		U	1	13
		R	1	15
		N	1	16
				53
.58	do	T	3/4	12
		U	3/4	17
		R	3/4	19
		N	3/4	22
				70

STRUCTURAL CHARACTERISTICS OF BUTTONS NOT STUDIED

The resolution of the American Association of State Highway Officials covered investigation of focal lengths as well as the spacing and optical diameters of the reflecting units. It was, however, found impracticable at this time to consider the method of backing, mounting, or geometric shapes of the reflectors because such investigations would involve reconstructing all test letters with buttons from each of a dozen or more responsible manufacturers and repeating all tests for each of the different commercial products. A complete set of letters and symbols requires approximately 2,000 reflector buttons. As indicated in this report, the prime consideration was to test the personal acuity of the average road user and the effect of a reflecting unit on the legibility of the sign message. The product of but one manufacturer was selected as representative of the industry. Because of this fact tests can be duplicated and the results of this investigation checked without introducing any variables not now considered. Moreover, tests of the character and quality of the materials used for metallic backing and method of sealing the reflector in its housing would involve exposure to the weather over a long period of time and definite conclusions could not be arrived at unless such exposure could be made in rural as well as congested urban areas, and the effect of gaseous contamination ascertained.

SPACING OF BUTTONS DISCUSSED

In spacing the individual reflecting buttons in a letter the first consideration obviously is clear definition of the letter, and this in turn depends, apparently, not so much on the optical diameter and spacing of the unit as upon factors which may cause a merging of parallel lines in such a way as to render the pattern indistinct or increase the glare to a point where the word cannot be recognized, resulting in the sign having what has been termed an arresting or signal value only. This phase is apparent in the percentages tabulated and plotted and particularly so where the use of the bent arrows in the CURVE and TURN signs tended to suggest to the observer the meaning without his being able to read the wording correctly.

Table 19 shows the number of units required for each letter and their spacing center to center, and figure 18 shows the general arrangement of the units in the letters S, R, and O. Table 20 shows the number and spacing of the buttons for the bent arrows used in the TURN and CURVE signs. It is believed that in these the minimum number of units that will adequately outline the letter is not exceeded and any attempt to reduce the number of reflectors in the interest of economy should not be permitted, because of the liability of a driver confusing, for instance, an ill-defined N with H or an ill-defined S with the figures 8 and 2.

TABLE 20.—*Number and spacing of reflector buttons used in bent arrows for STOP and SLOW signs*

Size of buttons	Width of arrow	Spacing of buttons		Number of buttons		
		Stem	Head cluster	Stem	Head	Total
Inches	*Inches*	*Inches*				
0.95	2 1/8	1 1/8	1 and 1 1/8 inche	9	4	13
.76	2 1/8	7/8	7/8 inch	11	10	21
.58	2 1/8	3/4	7/8 inch	14	10	24

CONCLUSIONS SUMMARIZED

In summing up the foregoing the various phases of the investigation will be treated in the order in which the tests were made.

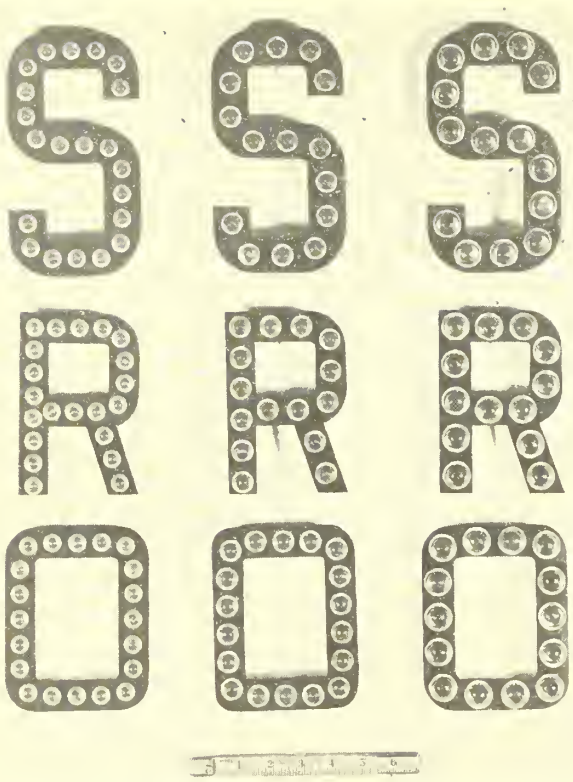

FIGURE 18.—ARRANGEMENT OF REFLECTOR BUTTONS IN LETTERS S, R, AND O.

The first phase involved determination of the relative values of the ground colors and the legibility of the respective legend colors and design. All evidence deduced and analyzed points to the definite conclusion that the standard yellow background with black design color, under all conditions which might reasonably be expected to obtain on either a city street or upon a rural highway, is much superior to black on white or white on black. Not only has this been demonstrated to be true as to readability of the legend but the yellow background is more arresting to the average observer, is more conspicuous by contrast with the average natural or artificial background, and, therefore, has a greater signal value than the other backgrounds or combinations. Moreover, by adopting a definite shade and hue, such as is covered by the specifications of the American Association of State Highway Officials in rule 53 of the sectional committee on colors for traffic signals of the American Standards Association, it may be possible by legislative act or otherwise to prohibit the use of this distinctive shade for advertising or for purposes other than official road signs upon the highways controlled by State or other properly constituted authorities. Obviously it would be difficult to obtain restrictive legislation for black or white backgrounds.

The investigation of the effect of headlights or local illumination such as street lights upon the signs not equipped with reflecting units was not in any way conclusive. When some of these observations were made the ground was covered with snow, of which there were several piles immediately adjacent to the line of sight, and conceivably this condition will obtain for several months in the year over a large area of the country. Such conditions emphasize the inadequacy of the daylight signs at night.

With respect to the use of reflecting units, there are two points to be considered which in a degree appear to conflict. In urban areas there is usually a great deal of illumination from street lights, electric signs, and other sources falling on the sign; and local regulations may require that headlights be dimmed or depressed. These conditions do not, as a rule, obtain on rural highways. In city driving speeds are relatively slower and clear perception of the sign message is more readily had than on rural highways. In view of these considerations, the conclusion is forced that the colorless button with a diameter of 0.76 inch with a minimum spacing of 1 inch, center to center, is the most efficient. This is notwithstanding the large increase in the efficiency of the 0.58-inch diameter button with dim headlights over the same button with full headlights. For rural use the small button apparently has a superior long-range signal value but is less readable than the unit of larger diameter.

The effect on distinctness in daylight of inserting reflecting buttons in the letters has been gone into exhaustively. As indicated by the tests, the effect of such insertions, even though approximately 50 percent of the superficial area of the letter is taken up by the use of colorless units, is not serious at 200 feet and the difference between black and white letters is negligible.

The recognition by the road user of the existence of a potential hazard by the shape or outline of the sign plaque alone has been briefly touched upon and the idea is susceptible of further development. Particularly is this true in view of the trend toward the use of symbols. The use of symbols, except for the arrows indicating curve or direction, is not considered as effective as outlining the sign with buttons. At night the road user can best be warned at long ranges against such hazards as may exist at narrow bridges, highway intersections, and other places in this category by clearly defining the shape of the plaque with reflecting units. Having been so warned he will be able to read the character of the hazard at 200 feet or well within braking distance for a car traveling at the maximum authorized speed.

These investigations had to do exclusively with the reactions of the average road user to the sign messages as measured by the tachistoscope. No account has been taken of the technical details of construction of the reflecting units, composition, refraction coefficients, or other optical properties of the glass; every effort, on the other hand, was made to simulate actual driving conditions. Maximum personal response to the warning is the ultimate consideration in sign design and it is not believed that optical analysis or other technical tests, however exhaustive, can measure this response. In the foregoing tests, 121 individuals were used as observers and these made a total of 7,710 separate observations involving about 12,000 items. These observers represent a fair cross section of the potential car-driving public. A number of them, however, had not received driving permits and were not car drivers. The gamut of education and personal intelligence was wide. A number of observers were furnished through the cooperation of the American Automobile Association, which in turn enlisted the aid of the local employment bureau. As indicated in the text, both males and females were used. Observations were made under the varied weather and seasonal conditions obtaining in the District of Columbia, and the results of the investigation may be taken as representative of the comparative visibility of the signs throughout the country as a whole.

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Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

AUGUST 31, 1933

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	Percentage completed	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$					$	$				$	
Alabama	2,385.7	4,215,785.67	2,107,892.74	89	97.0	103.1	200.1						3,157,514.70	Alabama
Arizona	1,426.7	913,963.20	352,587.93	78	26.9	29.5	56.4	74,368.32	51,819.92		8.7	8.7	106,364.30	Arizona
Arkansas	1,968.7	3,894,321.23	1,860,763.97	74	126.5	74.7	201.2	915,274.96	579,793.57	92.6	11.9	104.5	479,898.21	Arkansas
California	2,574.0	4,957,861.33	1,378,985.63	90	92.2	23.1	115.3						22,517.34	California
Colorado	1,890.6	2,691,086.13	1,165,919.30	62	90.5	46.5	137.0	173,962.64	78,283.16	2.2	21.5	23.7	47,698.17	Colorado
Connecticut	306.8	3,838,722.74	1,579,950.45	87	38.9		38.9						183,167.11	Connecticut
Delaware	413.5	138,879.50	25,512.72	64		7.3	7.3							Delaware
Florida	671.6	5,284,898.51	2,506,132.25	87	150.8	11.2	162.0		120,699.54				877,896.19	Florida
Georgia	3,332.9	3,010,067.23	1,239,254.11	93	89.6	104.3	193.9						100,636.93	Georgia
Idaho	1,607.7	2,241,411.93	834,741.56	84	73.4	116.8	190.2	113,905.40	51,929.06	4.0	24.0	28.0	12,987.00	Idaho
Illinois	3,177.2	18,141,194.46	6,697,500.54	81	561.0	56.6	617.6	486,007.26	231,323.05	17.5		17.5	119,376.28	Illinois
Indiana	2,173.5	7,763,205.12	2,589,042.73	88	239.5	24.9	264.4	13,439.70	2,167.84	.6		.6	19,336.79	Indiana
Iowa	3,721.6	1,842,467.88	324,092.99	89	109.4	1.1	110.5							Iowa
Kansas	4,083.4	3,155,204.24	900,986.34	77	186.8	46.3	233.1	400,368.33	109,578.66	72.4	68.0	140.4	20,489.02	Kansas
Kentucky	2,021.9	3,361,451.81	1,243,468.92	80	135.8	68.2	204.0	7,039.41	2,816.00		2.9	2.9	1.00	Kentucky
Louisiana	1,635.4	5,822,100.69	2,601,941.90	76	28.4	22.8	51.2						4,520.58	Louisiana
Maine	850.9	1,448,507.42	313,793.44	77	36.6	2.2	38.8						172.41	Maine
Maryland	881.9	749,787.77	95,954.66	88	27.2	1.5	28.7	112,967.80	12,639.48	2.4		2.4		Maryland
Massachusetts	876.4	4,115,901.96	972,218.48	88	57.2	4.9	62.1		229,375.00				169,445.05	Massachusetts
Michigan	2,486.9	4,633,278.33	1,819,467.45	79	227.1	37.0	264.1	202,500.00	95,835.00	3.6	5.5	9.1	16,266.12	Michigan
Minnesota	4,311.6	4,676,721.91	184,428.91	98	151.0	137.0	288.0						47,924.16	Minnesota
Mississippi	1,881.7	6,132,255.42	3,039,956.16	77	162.3	112.0	274.3	220,829.93	205,348.25	10.0	3.4	13.4	2,590,113.57	Mississippi
Missouri	3,261.9	3,841,847.52	672,265.63	71	135.7	33.7	169.4		50,048.60					Missouri
Montana	3,119.2	5,124,817.10	2,857,058.89	92	347.3	145.7	493.0	183,461.68	103,068.74	.5	53.3	53.8	146,648.28	Montana
Nebraska	4,478.1	1,815,058.75	842,092.22	94	57.7	31.7	89.4						163,468.22	Nebraska
Nevada	1,364.0	1,081,193.16	448,031.22	89	17.9	143.4	161.3	178,322.45	8,028.11	13.7	9.3	23.0	71,920.07	Nevada
New Hampshire	454.1	350,183.63	140,073.46	85	8.4	.5	8.9	175,656.85	77,387.63	1.8	2.3	4.1		New Hampshire
New Jersey	644.0	4,978,596.76	1,615,125.28	96	49.6		49.6						138,625.11	New Jersey
New Mexico	2,467.8	629,629.25	296,246.58	66	31.2	40.4	71.6	19,858.21	10,922.01		1.6	1.6	21,009.71	New Mexico
New York	3,551.4	17,103,830.05	5,042,650.00	73	444.3	32.0	476.3						355,781.37	New York
North Carolina	2,552.2	3,692,001.17	1,852,115.36	79	377.4	.5	377.9	129,408.73	407,098.37	11.9		11.9	641,264.98	North Carolina
North Dakota	5,619.9	2,700,986.74	1,348,048.63	61	174.5	329.3	503.8	244,282.00	114,051.47	11.5	57.4	68.9	205,902.91	North Dakota
Ohio	3,122.8	6,043,401.67	1,552,239.83	87	117.6	49.2	166.8	416,235.00	165,607.94	6.9	.3	7.2	13,739.73	Ohio
Oklahoma	2,547.1	3,417,500.89	1,103,939.12	90	162.1	62.0	224.1	1,093,216.00	434,320.36	69.7	5.6	75.3	98,134.21	Oklahoma
Oregon	1,655.5	3,535,967.28	1,483,611.28	88	94.8	59.3	154.1	17,877.20	7,329.65	.5		.5	53,738.69	Oregon
Pennsylvania	3,431.4	5,964,274.32	1,569,879.33	90	202.3	5.8	208.1						6,608.37	Pennsylvania
Rhode Island	287.8	435,012.32	120,817.09	91	6.6	4.3	10.9	18,249.80	9,124.90				61,709.13	Rhode Island
South Carolina	1,962.5	3,421,360.43	1,282,892.73	98	161.2	154.7	315.9		3,134.63				4,059.29	South Carolina
South Dakota	4,332.3	3,104,632.06	1,264,063.86	93	232.7	203.5	436.2	101,962.02	78,460.71		17.7	17.7	177,793.69	South Dakota
Tennessee	1,837.2	2,589,412.87	1,293,985.56	84	75.1	11.6	86.7		254,529.40				322,261.47	Tennessee
Texas	8,419.1	11,465,109.75	3,656,423.64	88	409.6	347.6	757.2	424,301.09	2,097.83		20.5	20.5	167,985.35	Texas
Utah	1,369.0	1,384,328.77	580,227.36	87	89.3	.2	89.5	118,531.26	49,939.43	17.1	11.8	28.9		Utah
Vermont	402.7	317,403.44	55,874.35	35	11.4	6.6	18.0	12,856.09	59.60	.1		.1	1,299.71	Vermont
Virginia	2,077.6	3,052,480.19	1,374,769.16	93	114.7	31.2	145.9	131,679.03	36,686.27	9.5		9.5	68,753.12	Virginia
Washington	1,340.0	2,774,263.82	781,920.63	88	92.9	16.9	109.8						3,356.82	Washington
West Virginia	937.5	2,574,911.01	1,057,657.50	94	95.2	14.7	109.9						20,322.07	West Virginia
Wisconsin	2,811.0	4,614,240.55	1,118,667.95	89	106.8	80.6	187.4		21,500.00					Wisconsin
Wyoming	2,360.6	1,578,401.84	604,430.49	96	111.3	59.8	171.1	134,390.27	67,100.00	6.2	12.8	19.0	73,262.89	Wyoming
Hawaii	139.3	420,141.08	420,141.08	36	7.3		7.3	87,664.50	29,902.55	.7		.7	485,864.71	Hawaii
TOTALS	111,226.6	191,040,060.90	68,269,841.41	83	6,443.0	2,896.2	9,339.2	6,208,615.93	3,702,006.73	355.4	338.5	693.9	11,299,834.83	TOTALS
Construction completed		159,354,000.00	57,005,000.00											
Balance uncompleted		31,686,000.00	11,265,000.00											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

A22.6

PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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G. P. St. CLAIR, *Editor*

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data ar obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

THE EFFECT OF VIBRATION AND DELAYED FINISHING ON PAVEMENT SLABS

Reported by F. H. JACKSON, Senior Engineer of Tests, and W. F. KELLERMANN, Associate Materials Engineer, Division of Tests, United States Bureau of Public Roads

IN THE August 1931 issue of PUBLIC ROADS there was published a report describing the results of an investigation conducted by the Bureau of Public Roads to determine the effect of certain controllable variables on the quality of pavement concrete. The test procedure used in this work differed from the conventional laboratory investigation in that observations and tests were made on actual pavement sections, placed and finished under working conditions, rather than on small-size test specimens. In this way it was possible to measure the effect of variations in workability by studying the quality and uniformity of the concrete in the finished structure rather than by attempting to investigate this indefinite property through the use of some arbitary laboratory test such as flow, penetration, etc.

SLUMP NOT LESS THAN 2 INCHES REQUIRED IN PRESENT CONSTRUCTION METHODS

In the tests which have already been reported a study was made of the effect of varying the quantity of coarse aggregate in the mix for various types and gradings of coarse aggregate and for various consistencies. The concrete was mixed and placed in accordance with accepted construction practice, which included finishing by the use of the conventional type of power-driven screeds, with and without tamper attachment. These initial tests developed certain information of value. For instance, it was quite definitely determined that with the methods of machine finishing now commonly employed a consistency corresponding to a slump of at least 2 inches must be used if honeycomb is to be avoided. Of course, such a concrete does not have as high a potential strength and density as a somewhat drier mix. Actually, however, it was found that both strength and uniformity were improved by the use of the wetter consistency. This result is explained by the fact that the segregation and honeycombing which accompanies the use of the drier mix actually tends to lower the strength of the pavement slab to a point considerably below the strength which would be expected theoretically from the water-cement ratio strength law. This means that under our present construction practice concretes mixed in the proportions used in paving work and with normal aggregates are not "workable" as paving mixes if the slumps are less than 2 inches. In other words, it is not possible to take advantage of the higher strength and density which should accompany the use of a lower water content than is required to give a 2-inch slump.

DEVELOPMENT OF METHODS FOR USING DRIER MIXES OBJECTIVE OF INVESTIGATION

A realization of this fact led naturally to a consideration of methods of placing which would permit the use of drier mixes without sacrificing strength or uniformity of strength. The use of vibratory screeds for this purpose presented interesting possibilities and it was decided to continue the experiments using a standard finishing machine equipped with vibrators mounted directly on the screeds. The present paper discusses the results of these tests, as well as another series in which a method of removing excess water from the concrete after it was deposited on the subgrade was investigated. It will be noted that in both cases the ultimate object was the same; that is, the production of a pavement slab of uniform quality with a lower net water content and consequently higher strength and greater density than would be possible with a standard finishing machine.

For the purpose of discussion the two series of tests are treated separately, the results obtained by the use of vibrators (series A) being presented first. This portion of the report is followed by a discussion of the tests which involved the principle of delayed finishing for the purpose of removing excess water, otherwise known as the Johnson method (series B). In both cases the general testing procedure outlined in the report of August 1931 was followed, except that the finishing operations were carried out in each case as indicated in detail below. The slabs were cast on the bare subgrade, whereas in the former tests the subgrade was covered with tar paper before placing the concrete.

The same general types of crushed limestone, gravel and blast-furnace slag were used as coarse aggregate. However, in the tests reported herein the aggregates were used in two separate sizes only instead of three, as in the earlier investigation. Two sands, differing considerably in gradation, were also used instead of one, as in the first series. In this way it was possible, in the case of series A, to investigate six combinations of aggregates varying as to character and gradation; that is, crushed stone, gravel, and slag, each in combination with a fine sand and a coarse sand. Blast furnace slag was omitted from the Johnson series and in its place was

TABLE 1.—*Properties of aggregates*

SIEVE[1] ANALYSIS

Total retained on—	Coarse sand	Fine sand	Gravel	Crushed stone	Crushed slag	Platte River combination
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1½-inch sieve			21	17	4	
¾-inch sieve			68	76	65	
⅜-inch sieve			90	95	99	11
No. 4 sieve	5	0	99	99	100	31
No. 8 sieve	24	2	100	100	100	46
No. 14 sieve	36	8	100	100	100	55
No. 28 sieve	54	30	100	100	100	67
No. 48 sieve	83	77	100	100	100	89
No. 100 sieve	96	96	100	100	100	98
Fineness modulus	2.98	2.13	7.78	7.87	7.68	3.97

PHYSICAL PROPERTIES

Approximate specific gravity	2.66	2.65	2.54	2.78	2.32	2.63
Absorption, percent	.82	.70	1.20	.36	1.25	.91
Decantation loss, percent	2.70	2.10				
Voids, percent[2]	37.00	41.00	32.00	40.00	43.00	31.00
Wear, percent[3]			9.60	3.40	7.70	

[1] Square openings.
[2] Dry-rodded.
[3] Abrasion tests conducted in accordance with methods applicable in each case.

substituted a sand-gravel combination conforming in grading to the so-called "Platte River gravel" used in Nebraska, Kansas, and Iowa. The essential test data pertaining to the aggregates used in this investigation are given in table 1. The same cement, a standard portland cement meeting all American Society for Testing Materials requirements, was used throughout. The proportions, consistency, etc., of the concrete for series A are shown in table 2 with similar data for series B in table 3.

TABLE 2.—*Data on mixes: Series A*

ROUND 1

Aggregates	Section no.[1]	Proportions by volume, dry-rodded	Cement factor in sacks per cubic yard	Water-cement ratio corrected	b/b_0	Slump
						Inches
A. Coarse sand and gravel	1-S	1:1.6:3.45	5.9	0.75	0.75	2¼
	2-V	1:1.6:3.45	5.9	.69	.76	1
	3-V	1:1.6:3.15	6.0	.65	.77	0
	4-V	1:1.6:3.70	5.7	.71	.78	2½
	5-V	1:1.6:3.95	5.5	.74	.80	1¼
	6-V	1:1.6:4.20	5.3	.73	.82	1
B. Fine sand and gravel	7-S	1:1.5:3.45	6.1	.73	.77	1½
	8-V	1:1.5:3.45	6.1	.68	.78	½
	9-V	1:1.5:3.45	6.1	.67	.78	¼
	10-V	1:1.5:3.70	5.8	.74	.80	1¼
	11-V	1:1.5:3.95	5.6	.73	.82	1
	12-V	1:1.5:4.20	5.5	.72	.85	0
C. Coarse sand and stone	13-S	1:1.9:3.30	6.1	.75	.75	1½
	14-V	1:1.9:3.30	6.2	.69	.76	¾
	15-V	1:1.9:3.30	6.2	.67	.76	0
	16-V	1:1.9:3.55	5.9	.75	.78	1½
	17-V	1:1.9:3.80	5.7	.74	.81	1½
	18-V	1:1.9:4.05	5.5	.74	.83	1¼
D. Fine sand and stone	19-S	1:1.7:3.30	6.4	.76	.78	1½
	20-V	1:1.7:3.30	6.5	.70	.79	1
	21-V	1:1.7:3.30	6.5	.68	.79	¾
	22-V	1:1.7:3.55	6.2	.75	.81	1¼
	23-V	1:1.7:3.80	6.0	.75	.84	1¼
	24-V	1:1.7:4.05	5.8	.75	.87	¾
E. Coarse sand and slag	25-S	1:2.1:3.20	6.2	.75	.73	2
	26-V	1:2.1:3.20	6.3	.67	.74	1
	27-V	1:2.1:3.20	6.3	.63	.75	½
	28-V	1:2.1:3.45	6.0	.73	.76	1¼
	29-V	1:2.1:3.70	5.8	.72	.80	1
	30-V	1:2.1:3.95	5.7	.70	.83	¾
F. Fine sand and slag	31-V	1:1.9:3.20	6.4	.79	.76	1
	32-S	1:1.9:3.20	6.3	.83	.75	2
	33-V	1:1.9:3.20	6.4	.75	.76	½
	34-V	1:1.9:3.45	6.1	.82	.78	1
	35-V	1:1.9:3.70	6.0	.81	.82	1
	36-V	1:1.9:3.95	5.8	.79	.85	½

ROUND 2

G. Coarse sand and gravel	37-S	1:1.6:3.45	6.0	0.67	0.76	2¼
	38-V	1:1.6:3.45	6.0	.61	.77	1
	39-V	1:1.6:3.45	6.1	.58	.78	¼
	40-V	1:1.6:3.70	5.8	.66	.79	1¼
	41-V	1:1.6:3.95	5.6	.66	.81	1¼
	42-V	1:1.6:4.20	5.4	.66	.84	1
H. Fine sand and gravel	43-V	1:1.5:3.45	6.1	.67	.78	1¼
	44-S	1:1.5:3.45	6.1	.72	.78	2½
	45-V	1:1.5:3.45	6.2	.61	.80	1
	46-V	1:1.5:3.70	5.9	.71	.80	2
	47-V	1:1.5:3.95	5.7	.71	.83	1½
	48-V	1:1.5:4.20	5.5	.70	.85	1¼
I. Coarse sand and stone	49-S	1:1.9:3.30	6.1	.75	.75	1¾
	50-V	1:1.9:3.30	6.2	.69	.76	1¼
	51-V	1:1.9:3.30	6.2	.65	.76	¼
	52-V	1:1.9:3.55	5.9	.75	.77	1¾
	53-V	1:1.9:3.80	5.7	.75	.80	1¾
	54-V	1:1.9:4.05	5.5	.75	.83	1
J. Fine sand and stone	55-S	1:1.7:3.30	6.4	.76	.78	2
	56-V	1:1.7:3.30	6.5	.70	.79	1½
	57-V	1:1.7:3.30	6.5	.66	.80	½
	58-V	1:1.7:3.55	6.2	.76	.81	2
	59-V	1:1.7:3.80	6.0	.76	.84	1½
	60-V	1:1.7:4.05	5.8	.76	.86	¼
K. Coarse sand and slag	61-S	1:2.1:3.20	6.2	.75	.73	2¾
	62-V	1:2.1:3.20	6.3	.66	.74	1½
	63-V	1:2.1:3.20	6.3	.62	.75	½
	64-V	1:2.1:3.45	6.0	.73	.75	1½
	65-V	1:2.1:3.70	5.8	.72	.80	1¼
	66-V	1:2.1:3.95	5.7	.70	.83	1

[1] Symbols used in this column, and in corresponding columns of subsequent tables have the following meanings: "S"—standard; "V"—vibrated; "J"—Johnson method.

TABLE 3—*Data on mixes—Series B*

ROUND 1

Aggregates	Section no.	Proportions by volume, dry-rodded	Cement factor in sacks per cubic yard	Water-cement ratio corrected	b/b_0	Slump
						Inches
A. Coarse sand and gravel	67-S	1:1.6:3.45	6.0	0.67	0.76	1
	68-J	1:1.6:3.45	5.9	.71	.76	2
	69-S	1:2.1:4.45	4.8	.76	.79	1
	70-J	1:2.1:4.45	4.8	.82	.79	2
B. Fine sand and gravel	71-S	1:1.5:3.45	6.1	.69	.78	1
	72-J	1:1.5:3.45	6.0	.76	.77	2
	73-S	1:2.0:4.45	4.8	.88	.80	1
	74-J	1:2.0:4.45	4.8	.95	.79	2
C. Coarse sand and stone	75-S	1:1.9:3.30	6.1	.75	.75	2
	76-J	1:1.9:3.30	6.1	.77	.74	2
	77-S	1:2.4:4.30	5.0	.87	.79	1
	78-J	1:2.4:4.30	4.9	.95	.78	3
D. Fine sand and stone	79-S	1:1.7:3.30	6.4	.76	.78	2
	80-J	1:1.7:3.30	6.3	.80	.77	3
	81-S	1:2.2:4.30	5.1	.96	.81	2
	82-J	1:2.2:4.30	5.1	.96	.81	2

ROUND 2

E. Coarse sand and gravel	83-S	1:1.6:3.45	6.0	0.67	0.76	2
	84-J	1:1.6:3.45	5.9	.71	.76	3
	85-S	1:2.1:4.45	4.8	.79	.79	2
	86-J	1:2.1:4.45	4.8	.82	.79	3
F. Fine sand and gravel	87-S	1:1.5:3.45	6.1	.72	.77	3
	88-J	1:1.5:3.45	6.1	.72	.77	2
	89-S	1:2.0:4.45	4.8	.87	.80	2
	90-J	1:2.0:4.45	4.8	.90	.79	3
G. Coarse sand and stone	91-S	1:1.9:3.30	6.1	.77	.74	2
	92-J	1:1.9:3.30	6.0	.80	.74	3
	93-S	1:2.4:4.30	4.9	.97	.77	2
	94-J	1:2.4:4.30	4.8	.99	.77	3
H. Fine sand and stone	95-S	1:1.7:3.30	6.4	.77	.78	2
	96-J	1:1.7:3.30	6.3	.79	.77	3
	97-S	1:2.2:4.30	5.1	.95	.81	2
	98-J	1:2.2:4.30	5.0	.98	.80	3
I. Platte River gravel	99-S	1:4	6.4	.93	--	1
	103-J	1:4	6.3	1.03	-----	2
	101-S	1:4.5	5.8	1.05	-----	2
	105-J	1:4.5	5.7	1.10	-----	2
J. Platte River gravel	100-S	1:4	6.4	.97	-----	1
	104-J	1:4	6.3	1.00	-----	2
	102-S	1:4.5	5.8	1.05	-----	1
	106-J	1:4.5	5.7	1.10	-----	2

TESTING PROCEDURE DESCRIBED

Strength tests.—As in the earlier tests, the flexura strength of each 9-foot pavement section was obtaine by averaging the results of flexure tests on four slab removed from the section and tested in accordance wit the method described in the first report. The actual te slabs were 27 inches in width, 60 inches in length, and inches in depth. They were loaded at the third points a 54-inch span. The same general procedure as regard control specimens was followed in the present series a in the previous tests. These specimens consisted beams tested at the age of 7 months and cylinders teste at the age of 9 months. These periods correspond to the ages at which the pavement slabs and cores drille therefrom were tested. All strength tests were mad on saturated specimens. The average results of a strength tests are shown in table 4. In table 5 are give the results of flexure tests on the individual slabs.

Absorption and density tests.—Four 6-inch cores we drilled from each test section, 2 each from 2 of the ha slabs remaining after the flexure tests. In drilling th cores considerable care was taken to select portions the slabs free from honeycomb in order to insure th use of sound, homogeneous cores for testing purpose Two of the cores, 1 from each slab, were tested in com pression while the other 2 were tested for absorptio and density. The following procedure was employe in making the absorption and density tests: The cor

TABLE 4.—*Results of strength tests*

Section no.	Modulus of rupture (in pounds per square inch): Pavement slabs at 7 months [1]	Modulus of rupture (in pounds per square inch): Control beams at 7 months [2]	Crushing strength (in pounds per square inch): Pavement cores at 9 months [3]	Crushing strength (in pounds per square inch): Control cylinders at 9 months [2]
1-S	737	578	6,240	1,910
2-V	712	658	6,620	5,700
3-V	624	566	6,950	4,800
4-V	780	692	6,460	5,100
5-V	689	682	6,680	5,100
6-V	648	709	6,640	4,280
7-S	743	656	6,880	4,700
8-V	711	604	7,040	5,550
9-V	691	643	6,760	5,590
10-V	714	629	6,650	4,590
11-V	659	638	6,490	4,840
12-V	606	562	6,230	5,180
13-S	852	892	6,780	6,410
14-V	917	892	7,580	6,870
15-V	880	858	7,550	6,980
16-V	961	864	7,460	6,740
17-V	944	845	7,540	6,350
18-V	895	868	7,590	6,510
19-S	890	857	6,240	6,520
20-V	905	853	7,450	7,020
21-V	863	894	7,040	6,920
22-V	1,006	928	5,980	6,380
23-V	914	915	7,080	6,150
24-V	813	852	6,740	5,590
25-S	802	704	6,540	5,960
26-V	762	744	6,170	6,680
27-V	678	722	6,730	6,910
28-V	780	696	6,810	6,680
29-V	751	708	6,300	5,920
30-V	807	730	6,340	6,380
31-V	754	678	6,500	6,120
32-S	730	680	6,450	6,180
33-V	764	682	6,620	5,820
34-V	815	716	5,900	6,100
35-V	824	720	6,480	6,100
36-V	727	706	6,210	5,860
37-S	697	705	6,740	5,870
38-V	735	684	6,740	5,960
39-V	642	631	8,300	1,800
40-V	714	632	7,180	4,980
41-V	675	646	6,690	5,710
42-V	669	615	7,020	5,890
43-V	692	640	5,640	5,060
44-S	723	624	7,270	6,110
45-V	730	598	7,540	6,120
46-V	720	659	6,910	5,450
47-V	708	607	6,960	5,180
48-V	664	700	6,800	6,020
49-S	854	819	7,630	6,580
50-V	961	914	8,240	7,380
51-V	828	776	8,040	5,780
52-V	949	862	7,080	7,030
53-V	1,010	909	7,920	6,620
54-V	877	834	7,450	6,300
55-S	908	812	6,980	5,880
56-V	965	811	7,300	7,140
57-V	947	819	7,760	6,440
58-V	989	900	7,240	6,480
59-V	962	803	7,160	6,740
60-V	951	824	7,330	6,610
61-S	802	742	6,550	6,170
62-V	818	734	7,230	6,560
63-V	860	762	7,490	6,760
64-V	862	678	6,620	5,160
65-V	825	726	6,980	6,310
66-V	819	715	6,670	6,160
67-S	724	642	7,280	6,760
68-J	777	664	7,800	6,080
69-S	658	645	6,930	5,870
70-J	680	604	6,830	5,780
71-S	666	683	6,380	5,800
72-J	753	587	7,070	5,960
73-S	628	602	5,900	4,960
74-J	681	543	6,340	4,800
75-S	884	862	7,710	6,440
76-J	918	865	8,220	6,890
77-S	709	756	6,040	6,100
78-J	908	698	8,060	5,370
79-S	934	836	7,320	7,010
80-J	1,062	801	8,280	6,700
81-S	784	670	5,940	5,180
82-J	910	688	7,180	5,280
83-S	688	668	7,140	6,230
84-J	724	638	7,690	5,580
85-S	672	572	5,660	5,180
86-J	697	598	6,640	4,690
87-S	712	666	6,400	5,920
88-J	756	667	7,040	5,720
89-S	643	566	5,160	4,800
90-J	660	518	6,000	4,640
91-S	849	860	7,520	6,590
92-J	934	775	8,000	5,830
93-S	814	703	5,700	4,660
94-J	882	622	7,220	4,040
95-S	819	858	7,060	6,720
96-J	978	807	8,000	6,640
97-S	771	694	5,500	5,300
98-J	811	628	7,050	4,280
99-S	584	601	4,900	4,380
103-J	729	598	5,910	4,580
101-S	637	564	4,200	4,260
105-J	680	500	5,720	3,720
100-S	694	640	4,740	4,900
104-J	728	626	6,060	4,750
102-S	638	534	4,080	3,960
106-J	700	599	5,810	3,930

[1] Each value average of 4 tests on 1 section.
[2] Each value average of 2 tests.
[3] Each value average of 2 tests on 1 section.

were first dried to constant weight in an electric oven where the temperature was maintained at approximately 150° C. They were then cooled, weighed, immersed in water and boiled for 5 hours. The specimens were then allowed to cool for 18 hours, after which they were removed from the water, surface-dried, and weighed. They were then weighed in water and from the three weights thus obtained the absorption, weight per cubic foot, and specific gravity were computed.

Values for the density of the hardened concrete were obtained by dividing the weight of a given volume of the concrete by the theoretical weight of solids in the same volume as calculated from the proportions and specific gravities of each of the constituent materials. In the case of series B, a value which may be termed the theoretical density of the plastic concrete was also obtained. This value is the ratio of the volume of solids (cement and aggregate) to the total volume of

TABLE 5.—*Individual slab strengths, uniformity, and honeycomb*

Section no.	Slab no.	Modulus of rupture (Lbs. per sq.in.)	Variation from average (Percent)	Honeycomb: Break (Percent)	Honeycomb: Bottom (Percent)
1-S	1	698	5.3	3.2	22.6
	2	738	.1	6	20.6
	3	766	3.9	0	5.1
	4	747	1.4	0	0
Average		737	2.7	1.0	12.1
2-V	1	694	2.5	0	4.3
	2	701	1.1	0	0
	3	740	3.9	0	0
	4	708	.6	.5	8.3
Average		712	2.0	.1	3.2
3-V	1	561	10.1	17.5	75.4
	2	590	5.4	12.5	62.1
	3	763	22.3	0	0
	4	580	7.1	13.5	55.8
Average		621	11.2	10.8	48.3
4-V	1	789	1.2	0	1.7
	2	742	4.9	0	0
	3	785	.6	0	0
	1	804	3.1	0	0
Average		780	2.4	0	.4
5-V	1	635	7.8	7.5	14.8
	2	660	4.2	0	4.4
	3	724	5.1	0	0
	4	738	7.1	0	0
Average		689	6.0	1.8	4.8
6-V	1	619	4.5	9.4	21.6
	2	623	3.9	0	0
	3	700	8.0	0	0
	4	649	.1	.5	2.7
Average		648	4.1	2.4	6.1
7-S	1	774	4.2	0	7.9
	2	700	5.8	0	11.1
	3	712	4.2	0	6.1
	4	785	5.7	0	1.9
Average		743	5.0	0	6.8
8-V	1	683	3.9	0	3.2
	2	753	5.9	0	0
	3	726	2.1	0	0
	4	682	4.1	1.5	16.2
Average		711	4.0	.4	4.8
9-V	1	684	1.0	3.8	25.8
	2	690	.1	0	0
	3	712	3.0	0	2.6
	4	678	1.9	2.1	16.6
Average		691	1.5	1.5	11.2
10-V	1	718	.6	0	0
	2	683	4.3	0	0
	3	743	4.1	0	0
	4	713	.1	0	0
Average		714	2.3	0	0
11-V	1	657	.3	0	4.7
	2	670	1.7	0	0
	3	680	3.2	0	0
	4	629	4.6	3.0	15.8
Average		659	2.4	.8	5.1
12-V	1	624	3.0	5.2	22.4
	2	660	8.9	0	0
	3	641	5.8	0	9.6
	4	499	17.7	7.1	74.5
Average		606	8.8	3.1	26.6
13-S	1	828	2.8	1.8	8.9
	2	793	6.9	1.6	26.8
	3	954	12.0	0	1.4
	4	831	2.5	1.8	7.0
Average		852	6.0	2.8	11.0
14-V	1	861	6.1	2.6	16.5
	2	917	0	0	0
	3	976	6.4	0	0
	4	915	.2	3.6	17.9
Average		917	3.2	1.6	8.6
15-V	1	790	10.2	6.3	14.4
	2	1,028	16.8	0	4.2
	3	979	11.2	0	0
	4	721	17.7	11.4	48.4
Average		880	14.0	5.1	16.8
16-V	1	988	2.8	0	0
	2	941	2.1	0	0
	3	985	2.5	0	0
	4	929	3.3	3.6	11.9
Average		961	2.7	.9	3.0
17-V	1	955	1.2	0	2.1
	2	963	2.0	0	0
	3	997	5.6	0	0
	4	861	8.8	4.6	10.2
Average		944	4.4	1.2	3.1
18-V	1	857	4.2	1.0	0
	2	801	10.5	4.3	11.4
	3	953	6.5	0	0
	4	968	8.2	3.6	24.9
Average		895	7.4	2.2	9.1
19-S	1	929	4.4	3.8	0
	2	775	12.9	4.2	30.2
	3	963	8.2	1.0	8.1
	4	895	.6	3.1	26.7
Average		890	6.5	3.0	16.2
20-V	1	868	4.1	5.3	30.2
	2	926	2.3	0	20.6
	3	1,039	14.8	0	0
	4	788	12.9	6.7	35.6
Average		905	8.5	3.0	21.6
21-V	1	858	.6	4.6	30.1
	2	878	1.7	2.6	33.4
	3	961	11.3	1.6	16.0
	4	756	12.4	8.2	70.4
Average		863	6.5	4.3	37.5
22-V	1	998	.8	0	0
	2	1,001	.5	0	0
	3	1,050	4.1	0	0
	4	975	3.1	1.5	2.8
Average		1,006	2.2	.4	.7
23-V	1	886	3.1	.5	4.5
	2	793	13.2	2.7	18.3
	3	1,036	13.3	0	0
	4	941	3.0	.5	0
Average		914	8.2	.9	5.8
24-V	1	817	.5	5.3	36.2
	2	820	.9	2.1	23.5
	3	889	9.3	0	0
	4	725	10.8	10.8	21.5
Average		813	5.4	4.5	20.3
25-S	1	873	8.9	0	3.2
	2	757	5.6	1.6	22.2
	3	807	.6	0	11.8
	1	771	3.9	0	13.0
Average		802	4.8	.4	12.6
26-V	1	738	3.1	1.1	17.0
	2	717	2.0	1.6	21.1
	3	818	7.3	0	3.4
	1	743	2.5	2.1	32.5
Average		762	3.7	1.2	18.6
27-V	1	654	3.5	6.6	10.4
	2	694	2.4	6.7	51.7
	3	748	10.3	2.6	50.1
	4	616	9.1	6.6	69.5
Average		678	6.3	5.6	52.9
28-V	1	795	1.9	0	0
	2	782	.3	0	7.9
	3	821	5.3	0	0
	4	722	7.1	0	4.5
Average		780	3.7	0	2.1
29-V	1	758	.9	0	1.2
	2	798	6.3	0	0
	3	796	6.0	0	0
	4	652	13.2	9.1	33.8
Average		751	6.6	2.3	8.8
30-V	1	761	5.7	3.8	15.9
	2	843	4.5	0	11.9
	3	885	9.7	0	7.4
	1	739	8.4	6.7	29.6
Average		807	7.1	2.6	16.4
31-V	1	797	5.7	0	0
	2	689	8.6	0	0
	3	746	1.1	0	0
	4	785	4.1	0	0
Average		754	4.9	0	0
32-S	1	854	17.0	0	0
	2	701	4.0	5.2	20.0
	3	738	1.1	3.1	11.7
	4	625	14.4	6.3	29.3
Average		730	9.1	3.7	15.2

TABLE 5.—*Individual slab strengths, uniformity, and honeycomb*—Continued

Section no.	Slab no.	Modulus of rupture	Variation from average	Honeycomb	
				Break	Bottom
		Lbs. per sq.in.	*Per-cent*	*Per-cent*	*Per-cent*
33-V	1	716	6.3	1.0	0.7
	2	754	1.3	.5	0
	3	759	.7	0	0
	4	828	8.4	0	1.0
Average		764	4.2	.4	.4
34-V	1	835	2.5	0	0
	2	827	1.5	0	0
	3	821	.7	0	0
	4	777	4.7	0	0
Average		815	2.4	0	0
35-V	1	841	2.1	0	0
	2	829	.6	0	0
	3	811	1.6	0	0
	4	814	1.2	0	6.5
Average		824	1.4	0	1.6
36-V	1	613	15.7	6.7	42.8
	2	728	.1	0	21.0
	3	833	14.6	0	0
	4	734	1.0	2.1	8.2
Average		727	7.8	2.2	18.0
37-S	1	683	2.0	0	0
	2	736	5.6	1.1	7.1
	3	696	.1	0	0
	4	674	3.3	0	0
Average		697	2.8	.3	1.8
38-V	1	657	10.6	3.7	30.9
	2	756	2.9	0	0
	3	814	10.7	0	0
	4	714	2.9	.5	0
Average		735	6.8	1.0	7.7
39-V	1	547	14.8	10.8	54.9
	2	642	0	1.1	29.7
	3	763	18.8	0	13.0
	4	615	4.2	5.3	32.8
Average		642	9.4	4.3	32.6
40-V	1	751	5.2	0	0
	2	720	.8	0	0
	3	659	7.7	0	0
	4	724	1.4	0	0
Average		714	3.8	0	0
41-V	1	672	.4	0	0
	2	692	2.5	0	0
	3	635	5.9	0	0
	4	700	3.7	0	0
Average		675	3.1	0	0
42-V	1	663	.9	2.5	13.5
	2	674	.7	1.6	8.7
	3	676	1.0	0	0
	4	664	0.7	1.1	20.4
Average		669	0.8	1.3	10.6
43-V	1	776	12.1	0	0
	2	672	2.9	0	0
	3	689	.4	0	0
	4	631	8.8	1.0	0
Average		692	6.0	.2	0
44-S	1	703	2.8	0	2.2
	2	711	1.7	0	3.6
	3	725	.3	0	0
	4	754	4.3	0	0
Average		723	2.3	0	1.4
45-V	1	736	.8	0	3.7
	2	720	1.4	0	1.5
	3	728	.3	0	0
	4	735	.7	0	0
Average		730	.8	0	1.3
46-V	1	774	7.5	0	0
	2	690	4.2	0	0
	3	735	2.1	0	0
	4	679	5.7	0	0
Average		720	4.9	0	0
47-V	1	741	4.7	0	0
	2	691	2.4	0	0
	3	701	1.0	0	0
	4	699	1.3	0	0
Average		708	2.4	0	0
48-V	1	667	.5	0	0
	2	593	10.7	2.6	3.1
	3	684	3.0	0	0
	4	710	6.9	0	0
Average		664	5.3	.6	.8
49-S	1	901	5.5	1.1	0
	2	757	11.4	5.7	44.9
	3	850	.5	1.0	1.5
	4	909	6.4	1.0	2.0
Average		854	6.0	2.2	12.1
50-V	1	936	2.6	0	0
	2	920	4.3	0	5.4
	3	1,017	5.8	0	0
	4	970	.9	0	0
Average		961	3.4	0	1.4
51-V	1	766	7.5	5.5	32.6
	2	745	10.0	6.2	54.8
	3	912	10.1	0	10.3
	4	890	7.5	1.1	16.3
Average		828	8.8	3.9	28.5
52-V	1	954	.5	0	0
	2	880	7.3	3.1	0
	3	1,029	8.4	0	0
	4	932	1.8	0	0
Average		949	4.5	.8	0
53-V	1	992	1.8	0	0
	2	1,017	.7	0	5.6
	3	1,067	5.6	0	0
	1	962	4.7	0	0
Average		1,010	3.2	0	1.4
54-V	1	834	4.9	1.6	14.0
	2	854	2.6	2.1	20.5
	3	964	9.9	0	0
	1	857	2.3	1.6	5.9
Average		877	1.9	1.3	10.1
55-S	1	952	4.9	0	0
	2	863	5.0	.5	18.9
	3	936	3.1	0	10.9
	4	879	3.2	0	5.7
Average		908	4.0	.1	8.9
56-V	1	955	1.0	0	0
	2	959	.6	.5	4.2
	3	948	1.8	0	0
	1	998	3.4	0	0
Average		965	1.7	.1	1.0
57-V	1	1,010	6.7	0	0
	2	884	6.6	4.3	28.7
	3	976	3.1	0	0
	4	917	3.2	1.5	5.2
Average		947	4.9	1.4	8.5
58-V	1	1,006	1.7	0	0
	2	945	4.4	0	0
	3	1,024	3.5	0	0
	4	982	.7	0	0
Average		989	2.6	0	0
59-V	1	972	1.0	0	0
	2	930	3.3	0	0
	3	976	1.5	0	0
	4	970	.8	0	0
Average		962	1.6	0	0
60-V	1	871	8.4	0	0
	2	884	7.4	0	0
	3	1,151	21.0	0	0
	4	900	5.4	0	0
Average		951	10.6	0	0
61-S	1	767	4.4	0	0
	2	793	1.1	0	3.5
	3	794	1.0	0	3.3
	4	854	6.5	0	0
Average		802	3.2	0	1.7
62-V	1	[1] 462	....	0	0
	2	[1] 220	....	0	0
	3	833	1.8	0	0
	1	802	2.0	0	0
Average		818	1.9	0	0
63-V	1	858	.2	0	0
	2	889	3.4	0	0
	3	856	.5	0	0
	4	837	2.7	1.0	2.7
Average		860	1.7	.2	.7
64-V	1	867	.6	0	0
	2	857	.6	0	0
	3	882	2.3	0	0
	4	841	2.4	0	0
Average		862	1.5	0	0

[1] Not included in average.

TABLE 5.—*Individual slab strengths, uniformity, and honeycomb*—Continued

Section no.	Slab no.	Modulus of rupture	Variation from average	Honeycomb	
				Break	Bottom
		Lbs. per sq.in.	*Per-cent*	*Per-cent*	*Per-cent*
65-V	1	860	4.2	0	0
	2	813	1.5	0	0
	3	838	1.6	0	0
	4	790	4.2	2.0	2.8
Average		825	2.9	.5	.7
66-V	1	828	1.1	0	0
	2	831	1.5	0	0
	3	843	2.9	0	0
	4	775	5.4	3.1	4.3
Average		819	2.7	.8	1.1
67-S	1	789	9.0	0	3.0
	2	703	2.9	4.2	27.7
	3	694	4.1	0	0
	1	712	1.7	.5	4.6
Average		724	4.4	1.1	8.8
68-J	1	829	6.7	0	0
	2	737	5.1	2.7	19.9
	3	787	1.3	0	0
	4	756	2.7	2.1	15.2
Average		777	4.0	1.2	8.8
69-S	1	703	6.8	3.6	1.7
	2	618	6.1	4.2	28.4
	3	645	2.0	7.3	4.1
	4	668	1.5	2.0	13.4
Average		658	4.1	1.3	11.9
70-J	1	723	6.3	0	0
	2	684	.6	2.6	22.1
	3	633	6.9	.5	0
	4	681	.1	2.1	4.0
Average		680	3.5	1.3	6.5
71-S	1	701	5.3	.5	4.3
	2	611	8.3	3.6	57.2
	3	690	3.6	1.0	9.2
	4	661	.8	3.0	30.9
Average		666	4.5	2.0	25.4
72-J	1	711	5.6	0	0
	2	743	1.3	0	0
	3	812	7.8	0	3.7
	4	747	.8	0	0
Average		753	3.9	0	.9
73-S	1	598	4.8	3.1	3.2
	2	600	4.5	2.1	25.6
	3	660	5.1	0	0
	4	652	3.8	0	0
Average		628	4.6	1.3	7.2
74-J	1	655	3.8	0	0
	2	678	.4	1.6	9.4
	3	708	4.0	0	0
	4	684	.4	0	0
Average		681	2.2	.4	2.4
75-S	1	994	12.4	0	0
	2	842	4.8	5.8	23.5
	3	931	5.3	0	2.0
	4	771	12.8	5.2	3.7
Average		884	8.8	2.7	7.3
76-J	1	915	.3	0	0
	2	942	2.6	0	0
	3	908	1.1	0	0
	4	905	1.4	2.5	12.5
Average		918	1.4	.6	3.1
77-S	1	831	17.2	2.5	9.4
	2	745	5.1	11.3	36.9
	3	551	22.3	19.1	22.4
	4	710	.1	10.7	21.4
Average		709	11.2	10.9	22.5
78-J	1	818	9.9	0	0
	2	965	6.3	0	0
	3	977	7.6	0	0
	4	871	4.1	0	3.1
Average		908	7.0	0	.8
79-S	1	918	1.7	0	0
	2	870	6.8	2.1	9.5
	3	959	2.7	0	6.0
	4	987	5.7	0	3.4
Average		934	4.2	.5	4.7
80-J	1	1,096	3.2	0	0
	2	1,030	3.0	0	3
	3	1,075	1.2	0	0
	4	1,048	1.3	0	0
Average		1,062	2.2	0	.8
81-S	1	842	7.4	0	0
	2	617	21.3	10.3	15.
	3	875	11.6	0	0
	4	802	2.3	2.1	1.
Average		784	10.6	3.1	4.
82-J	1	904	.7	0	0
	2	900	1.1	.5	4
	3	921	1.2	0	0
	4	917	.8	0	0
Average		910	1.0	.1	1
83-S	1	666	3.2	2.0	15
	2	678	1.5	1.0	20
	3	685	.4	3.8	27
	4	721	4.8	2.1	3
Average		688	2.5	2.2	16
84-J	1	696	3.9	0	0
	2	698	3.6	0	0
	3	753	4.0	.5	10
	4	748	3.3	0	0
Average		724	3.7	.1	2
85-S	1	638	5.1	0	0
	2	699	4.0	0	0
	3	674	.3	1.6	2
	4	678	.9	0	
Average		672	2.6	.4	
86-J	1	735	5.2	0	0
	2	711	2.0	0	0
	3	667	4.3	0	0
	4	678	2.7	0	0
Average		697	3.6	0	0
87-S	1	720	1.1	0	5
	2	709	.4	0	5
	3	683	4.1	0	4
	4	735	3.2	0	0
Average		712	2.2	0	3
88-J	1	733	3.0	1.1	0
	2	785	3.8	0	0
	3	765	1.2	1.0	2
	4	740	2.1	2.5	7
Average		756	2.5	1.2	2
89-S	1	612	4.8	1.0	0
	2	621	3.4	0	0
	3	651	1.2	.5	11
	4	688	7.0	0	0
Average		643	4.1	.4	3
90-J	1	654	.9	.5	
	2	665	.8	0	0
	3	670	1.5	.5	10
	4	653	1.1	0	4
Average		660	1.1	.3	3
91-S	1	924	8.8	0	0
	2	874	2.9	0	4
	3	738	13.1	12.9	23
	4	860	1.3	2.0	0
Average		849	6.5	3.7	6
92-J	1	910	2.6	0	0
	2	948	1.5	0	0
	3	906	3.0	1.5	4
	4	974	4.3	0	
Average		934	2.8	.4	1
93-S	1	794	2.5	0	0
	2	890	9.3	0	0
	3	760	6.6	3.3	17
	4	812	.2	1.0	
Average		814	4.6	1.8	4
94-J	1	872	1.1	0	0
	2	928	5.2	0	0
	3	918	4.1	0	0
	4	810	8.2	0	0
Average		882	4.6	0	0
95-S	1	892	5.1	.5	8
	2	884	4.1	0	11
	3	785	7.5	3.5	26
	4	834	1.8	1.0	0
Average		849	4.6	1.3	11
96-J	1	1,018	4.1	0	0
	2	1,016	3.9	0	0
	3	1,018	4.1	0	2
	4	862	11.9	0	0
Average		978	6.0	0	0

TABLE 5.—*Individual slab strengths, uniformity, and honeycomb* Continued

Section No.	Slab No.	Modulus of rupture	Variation from average	Honeycomb	
				Break	Bottom
		Lbs. per sq.in.	*Per cent*	*Per cent*	*Per cent*
97-S	1	826	7.1	0	7.9
	2	832	7.9	0	10.3
	3	636	17.5	10.6	38.5
	4	789	2.3	2.7	2.2
Average		771	8.7	3.2	14.7
98-J	1	852	5.1	0	2.9
	2	873	7.6	0	0
	3	725	10.6	7.9	23.9
	4	795	2.0	.5	4.1
Average		811	6.3	2.1	7.7
99-S	1	552	5.5	2.5	0
	2	532	8.9	2.5	0
	3	581	.5	.5	0
	4	670	14.7	0	0
Average		584	7.4	1.4	0
100-S	1	689	.7	0	0
	2	677	2.4	0	0
	3	701	1.0	0	0
	4	708	2.0	0	0
Average		694	1.5	0	0
101-S	1	618	3.0	0	0
	2	639	.3	0	0
	3	634	.5	0	0
	4	657	3.1	0	0
Average		637	1.7	0	0
102-S	1	629	1.4	0	0
	2	649	4.7	0	0
	3	620	2.8	0	0
	4	654	2.5	0	0
Average		638	2.1	0	0
103-J	1	747	1.6	0	0
	2	723	.8	0	0
	3	754	3.1	.5	0
	4	721	1.1	0	0
Average		729	1.7	1	0
104-J	1	699	4.0	0	0
	2	763	4.8	0	0
	3	742	1.9	.5	0
	4	706	3.0	0	0
Average		728	3.4	1	0
105-J	1	634	6.8	0	0
	2	704	3.5	0	0
	3	714	5.0	0	0
	4	667	1.9	0	0
Average		680	4.3	0	0
106-J	1	705	.7	0	0
	2	750	7.1	0	0
	3	677	3.3	0	0
	4	668	4.6	0	0
Average		700	3.9	0	0

concrete calculated on the assumption that the freshly mixed concrete is free from air voids. This, of course, is not absolutely true. However, tests of concrete similar to that used in series B, where the difference in consistency of the concretes finished by the two methods was not great (ranging from about 1½ inches to 3 inches in slump), have shown that the error introduced by not correcting for air voids is small and reasonably constant and therefore may be neglected. On this basis a comparison of the two values for density may be used as a rough measure of the amount of water loss taking place during consolidation. In other words, it may be assumed that the increase in density of the hardened concrete as compared with the density of the plastic concrete is due to the elimination of water only. The significance of these values is discussed in that portion of the report dealing with series B.

Values for absorption, apparent specific gravity, and final density are shown in table 6.

Determination of honeycomb.—The extent and distribution of visible honeycomb in the test slabs was carefully determined by measurement of the fractured slabs remaining after the flexure test. The actual area of honeycomb on the bottom of each 27-inch by 60-inch slab was determined by means of a wire grid having 1-inch square openings laid upon the surface. (See fig. 1.) The amount of honeycomb in the cross section at the break was determined by plotting the honeycomb area on cross-section paper. These procedures provided an accurate method of determining the percentage of the bottom surface which contained honeycomb areas exceeding approximately one fourth inch in depth and the percentage of honeycomb at the break. The average percentage of honeycomb for an entire section as used in the discussion is the average of 8 determinations, 1 on the bottom and 1 at the plane of fracture for each of the 4 test slabs composing the section. The results of all determinations of honeycomb are shown in table 5, in connection with the results of flexure tests on the individual slabs.

TABLE 6. *Specific gravity, density, and absorption tests of cores*

Section no. and finish	Specific gravity			Density			Absorption		
	(1)	(2)	Average	(1)	(2)	Average	(1)	(2)	Average
				Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1-S	2.32	2.31	2.32	88.1	87.7	87.9	5.00	5.14	5.07
2-V	2.34	2.31	2.32	88.8	87.7	88.2	4.07	5.23	4.65
3-V	2.32	2.33	2.32	88.1	88.4	88.2	5.19	4.16	4.82
4-V	2.33	2.31	2.32	88.5	87.8	88.2	5.01	5.35	5.18
5-V	2.37	2.34	2.34	90.2	87.9	89.0	4.10	5.20	4.65
6-V	2.35	2.32	2.34	89.5	88.1	89.0	4.31	4.96	4.64
7-S	2.28	2.28	2.28	86.4	86.1	86.1	5.57	5.73	5.65
8-V	2.28	2.31	2.30	86.4	87.6	87.0	5.42	5.10	5.26
9-V	2.29	2.30	2.30	86.8	87.2	87.0	4.70	4.86	4.78
10-V	2.27	2.30	2.28	86.2	87.4	86.8	5.96	5.06	5.51
11-V	2.33	2.28	2.30	88.6	86.7	87.6	4.93	5.13	5.03
12-V	2.28	2.33	2.30	86.8	88.7	87.8	5.18	4.61	4.91
13-S	2.35	2.36	2.36	81.8	85.1	85.0	5.40	5.21	5.30
14-V	2.36	2.38	2.37	85.1	85.9	85.5	5.16	4.38	4.77
15-V	2.36	2.38	2.37	85.1	85.9	85.5	4.97	4.70	4.84
16-V	2.38	2.37	2.38	85.9	85.5	85.7	5.10	5.18	5.14
17-V	2.38	2.36	2.37	85.9	85.1	85.5	4.97	4.91	4.94
18-V	2.39	2.39	2.39	86.2	86.2	86.2	4.38	4.72	4.55
19-S	2.36	2.38	2.37	84.9	85.6	85.2	5.44	5.01	5.22
20-V	2.37	2.38	2.38	85.3	85.6	85.4	4.66	4.66	4.66
21-V	2.38	2.37	2.38	85.6	85.5	85.4	4.63	4.96	4.80
22-V	2.39	2.38	2.38	86.0	85.6	85.8	4.93	5.07	5.00
23-V	2.36	2.39	2.38	84.9	86.0	85.1	5.00	5.03	5.02
24-V	2.38	2.38	2.38	85.6	85.6	85.6	4.74	4.61	4.69
25-S	2.15	2.26	2.20	84.2	88.5	86.1	7.38	6.70	7.01
26-V	2.16	2.19	2.18	84.6	85.8	85.2	6.99	6.37	6.68
27-V	2.16	2.16	2.16	84.6	84.6	84.6	6.97	6.63	6.80
28-V	2.18	2.19	2.18	86.0	86.1	86.2	6.71	6.13	6.37
29-V	2.16	2.17	2.16	85.3	85.7	85.5	6.94	6.59	6.76
30-V	2.17	2.20	2.18	85.9	87.4	86.5	6.96	6.31	6.61
31-V	2.14	2.15	2.14	84.1	84.5	84.3	8.10	7.98	8.04
32-S	2.15	2.13	2.14	84.5	83.7	84.1	8.36	8.09	8.22
33-V	2.14	2.17	2.16	84.1	85.3	84.7	7.51	7.16	7.18
34-V	2.12	2.16	2.14	83.9	85.5	84.7	8.19	7.97	8.08
35-V	2.15	2.13	2.14	85.3	84.5	84.9	7.94	8.24	8.09
36-V	2.14	2.14	2.11	85.0	85.0	85.0	7.86	7.61	7.74
37-S	2.32	2.29	2.30	88.1	86.9	87.5	4.77	5.13	4.95
38-V	2.32	2.32	2.32	88.1	88.1	88.4	4.81	4.83	4.82
39-V	2.31	2.33	2.32	87.7	88.4	88.0	4.84	4.28	4.56
40-V	2.33	2.33	2.33	88.5	88.5	88.5	4.26	4.35	4.30
41-V	2.35	2.33	2.34	89.4	88.7	89.0	3.83	4.32	4.08
42-V	2.37	2.36	2.36	90.3	89.9	90.1	3.95	3.62	3.78
43-V	2.29	2.32	2.30	86.8	88.0	87.4	5.30	4.88	5.09
44-S	2.28	2.29	2.28	86.4	86.8	86.6	5.47	5.28	5.38
45-V	2.33	2.30	2.32	88.3	87.2	87.8	4.84	5.07	4.96
46-V	2.32	2.32	2.32	88.1	88.1	88.1	4.94	4.74	4.84
47-V	2.32	2.32	2.32	88.2	88.2	88.2	5.12	4.83	4.98
48-V	2.33	2.33	2.33	88.7	88.7	88.7	4.70	4.75	4.72
49-S	2.37	2.38	2.38	85.5	85.9	85.7	4.44	4.99	4.72
50-V	2.41	2.41	2.41	86.9	86.9	86.9	4.40	3.77	4.08
51-V	2.38	2.39	2.38	85.9	86.2	86.0	4.86	4.53	4.70
52-V	2.38	2.40	2.39	85.9	86.6	86.2	4.80	4.64	4.72
53-V	2.38	2.42	2.40	85.9	87.3	86.6	4.82	3.92	4.37
54-V	2.42	2.42	2.42	87.3	87.3	87.3	4.32	3.84	4.08
55-S	2.34	2.38	2.36	84.2	85.6	84.9	5.80	4.56	5.18
56-V	2.37	2.37	2.37	85.3	85.3	85.3	5.05	4.64	4.84
57-V	2.36	2.37	2.36	84.9	85.3	85.1	5.08	5.10	5.09
58-V	2.37	2.37	2.37	85.3	85.3	85.3	5.31	5.31	5.31
59-V	2.39	2.40	2.40	86.0	86.3	86.2	4.48	5.04	4.76
60-V	2.42	2.40	2.41	87.1	86.3	86.7	4.37	4.63	4.50
61-S	2.16	2.14	2.15	84.6	83.9	84.2	7.91	8.32	8.12
62-V	2.17	2.16	2.16	85.0	84.6	84.8	7.36	7.29	7.32
63-V	2.21	2.17	2.19	86.6	85.0	85.8	6.89	7.03	6.96
64-V	2.19	2.18	2.18	86.4	86.0	86.2	7.41	7.34	7.38
65-V	2.19	2.17	2.18	86.5	85.7	86.1	7.24	7.21	7.22
66-V	2.16	2.18	2.17	85.5	86.3	85.9	7.14	6.87	7.00
67-S	2.30	2.31	2.30	87.3	87.7	87.5	5.28	5.02	5.15
68-J	2.36	2.33	2.34	89.6	88.4	89.0	4.14	4.88	4.70
69-S	2.31	2.35	2.33	88.0	89.5	88.8	4.77	4.15	4.60
70-J	2.32	2.36	2.34	88.4	89.9	89.2	4.75	4.35	4.55
71-S	2.28	2.32	2.30	86.4	88.0	87.2	5.26	4.90	5.08
72-J	2.31	2.30	2.30	87.6	87.2	87.4	5.20	5.36	5.28
73-S	2.24	2.30	2.27	85.4	87.7	86.6	5.79	5.18	5.48
74-J	2.31	2.31	2.31	88.0	88.0	88.0	5.02	5.01	5.02
75-S	2.38	2.40	2.39	85.8	86.6	86.2	5.04	4.79	4.92
76-J	2.39	2.40	2.40	86.2	86.6	86.4	4.45	4.70	4.42
77-S	2.37	2.37	2.37	85.6	85.6	85.6	4.74	5.22	4.98
78-J	2.39	2.40	2.40	86.4	86.7	86.6	4.90	4.87	4.88
79-S	2.37	2.34	2.36	85.2	84.2	84.7	5.54	5.95	5.74
80-J	2.40	2.37	2.38	86.3	85.2	85.8	4.69	5.45	5.07
81-S	2.35	2.34	2.34	84.9	84.5	84.7	5.55	5.75	5.65
82-J	2.37	2.39	2.38	85.6	86.3	86.0	5.17	4.97	5.07
83-S	2.29	2.33	2.31	86.9	88.4	87.6	5.37	5.44	5.40
84-J	2.34	2.36	2.35	88.8	89.6	89.2	4.59	3.82	4.20
85-S	2.31	2.29	2.30	88.0	87.2	87.6	4.97	5.23	5.10
86-J	2.30	2.36	2.33	87.6	89.9	88.8	5.30	3.85	4.58
87-S	2.32	2.28	2.30	88.0	86.4	87.2	4.66	5.39	5.02
88-J	2.29	2.35	2.32	86.8	89.1	88.0	5.43	3.93	4.68
89-S	2.29	2.28	2.28	87.3	86.9	87.1	5.14	5.39	5.25
90-J	2.27	2.31	2.29	86.5	88.0	87.2	5.55	5.11	5.33
91-S	2.35	2.39	2.37	84.8	86.2	85.5	5.42	4.29	4.86
92-J	2.38	2.43	2.40	85.8	87.7	86.8	4.77	4.20	4.48
93-S	2.38	2.39	2.38	86.0	86.4	86.2	3.92	4.09	4.00
94-J	2.40	2.38	2.39	86.7	86.0	86.4	4.03	4.89	4.46
95-S	2.36	2.36	2.36	84.9	84.9	84.9	5.51	5.40	5.46
96-J	2.36	2.39	2.38	84.9	86.0	85.4	5.30	4.84	5.07
97-S	2.31	2.35	2.33	83.4	84.9	84.2	6.05	5.63	5.84
98-J	2.36	2.37	2.36	85.2	85.6	85.4	5.19	5.26	5.22
99-S	2.08	2.08	2.08	77.2	77.2	77.2	8.46	7.97	8.06
103-J	2.10	2.13	2.12	78.0	79.1	78.6	7.78	7.46	7.62
101-S	2.08	2.08	2.08	77.4	77.4	77.4	8.11	8.09	8.10
105-J	2.14	2.14	2.14	79.6	79.6	79.6	7.17	7.02	7.10
100-S	2.09	2.07	2.08	77.6	76.8	77.2	7.80	8.56	8.18
104-J	2.12	2.14	2.13	78.7	79.4	79.0	7.40	7.03	7.22
102-S	2.06	2.06	2.06	76.6	76.6	76.6	8.43	7.90	8.16
106-J	2.15	2.15	2.15	80.0	80.0	80.0	6.64	7.09	6.86

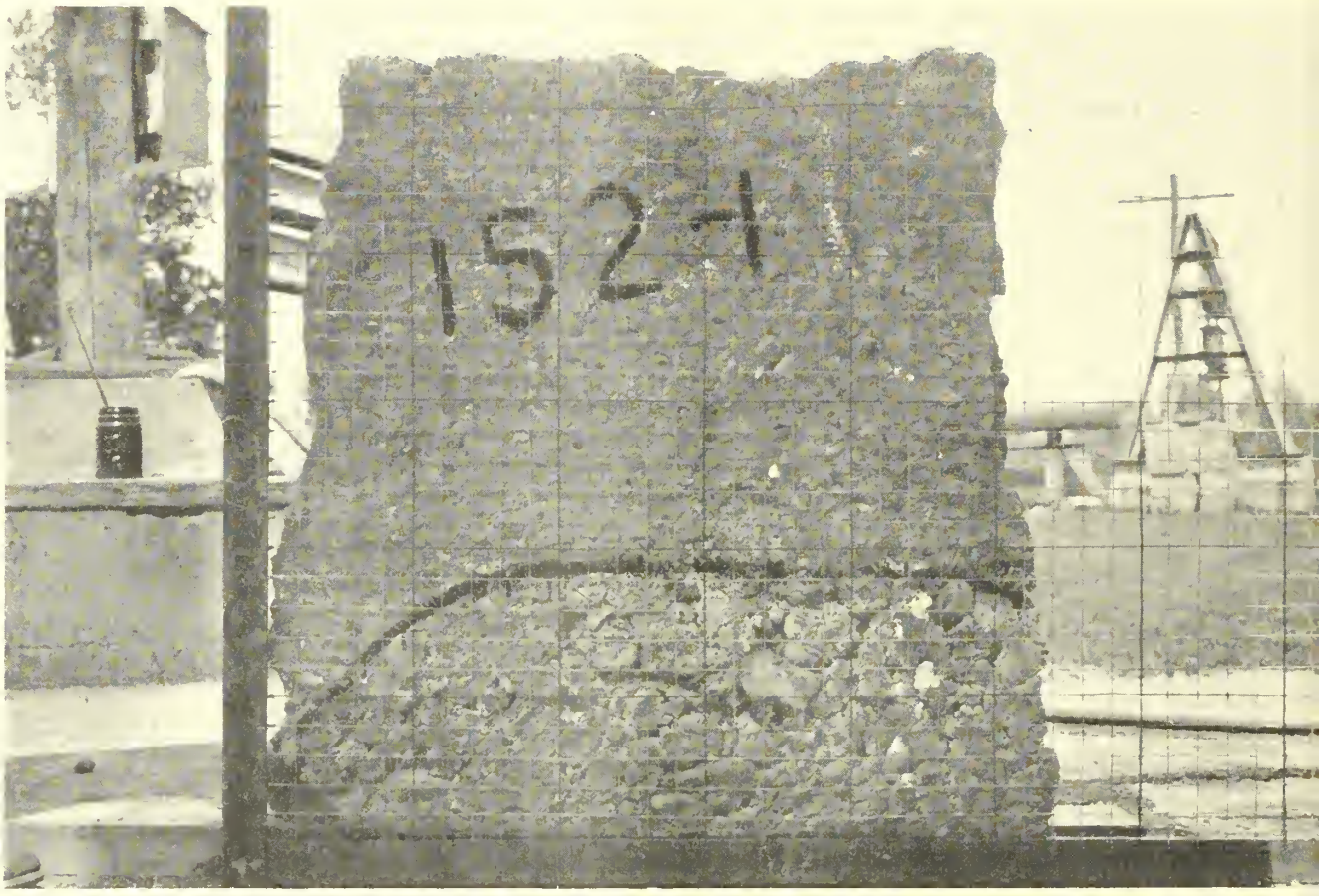


FIGURE 1.—ILLUSTRATING USE OF GRID WITH 1-INCH SQUARE OPENINGS FOR MEASURING HONEYCOMB.

Determination of surface wear.— Determinations of the surface hardness of the test slabs were made with a special device designed by the Bureau and described in detail in Public Roads for July 1929. The hardness testing machine consists essentially of a vertical shaft carrying a frame upon which are mounted three steel wheels. The wheels roll on the concrete in a circular path 21 inches in diameter and gradually wear a narrow groove in the surface, the depth of which, after a given number of revolutions, is used as a measure of the hardness of the mortar surface. Care is taken to see that the wear is confined to the mortar surface only so as to avoid variations due to any difference in hardness of the mortar and the coarse aggregate. (See fig. 2.)

The results of the strength tests, together with data showing the amount of honeycombing existing in each test slab, the density, absorption, and resistance to wear of the concrete, form the basis of comparison used in this report to determine the effect of the methods of finishing on quality.

In all, one hundred and six 9-foot sections were cast during this investigation, 66 in connection with the investigation of the effect of vibration and 40 in connection with the tests of the Johnson method. The first slabs were cast on September 10, 1931, and the last on October 5, 1931, progress being at the rate of 6 slabs per day for the vibration series and 8 slabs per day for the Johnson series.

FIGURE 2.—GROOVE WORN IN SURFACE AFTER 600 REVOLUTIONS OF HARDNESS TESTING MACHINE; CIRCLE 21 INCH IN DIAMETER.

SERIES A: TESTS TO DETERMINE EFFECT OF VIBRATION

For this series of tests a standard double-screed finishing machine was equipped with 3 high-frequency vibrators, 2 mounted on the front screed and 1 on the rear screed. (See fig. 4.) Each vibrator consisted of a ⅓-horsepower electric motor operating at an approximate speed of 3,600 revolutions per minute. There was attached to the rotor of each motor an eccentric weight which, revolving at high speed, imparted a vibratory motion to the screed. The motors were operated with current supplied from a gasoline-electric generator unit mounted on the finishing machine.

The only modification of the finishing machine as ordinarily used, other than the installation of the vibrators, was the use of a "bull-nose" strike-off which was riveted to the front screed. A view of the finishing machine showing the special strike-off is shown in figure 3. This device was furnished and recommended by the manufacturer of the finishing machine on the theory that the rounded surface of the "bull-nose" would tend to consolidate as well as strike off the surface of the concrete.

In general, the finishing machine was operated at about the same forward speed as is customarily used. The number of passes was limited to three as in the former tests, this being considered the maximum allowable from the standpoint of production. However, it was found necessary to exceed two passes in only five cases.

With the set-up as described above the vibrations could be distinctly felt by a person standing on the ground outside the line of forms. Attention, however, should be called to the fact that the usual standard-weight screeds were employed in this work. These screeds with attachments weigh approximately the same per foot of length for various widths of road. It is probable that a somewhat greater vibratory effect would have been obtained had the machine been equipped with lighter screeds. A somewhat greater effect would probably have been obtained, even with the standard screeds, had an 18- or 20-foot section been tested instead of a 9-foot section.

DRIER MIXES SATISFACTORY IF CONCRETE IS VIBRATED

Results of tests in which vibrated concrete is compared with concrete placed by the standard method lead to the following conclusions:

1. By the use of vibrating equipment of the same general type as employed in connection with these tests it should be possible satisfactorily to place and finish considerably drier concrete than is possible with methods now in common use.

2. For conditions comparable to these tests, it should be possible satisfactorily to place and finish by vibration concrete having a minimum slump of 1 inch as compared to a minimum slump of 2½ inches for methods now in common use.

3. These tests indicate that, depending on the type of coarse aggregate used, concrete containing from one fourth to three fourths part more coarse aggregate than the base mix, if vibrated, should show as great uniformity and as high flexural strength as the base mix finished by methods now in common use. Such a mix will contain approximately from 0.2 to 0.6 sack of cement per cubic yard less than the base mix.

4. The indications are that the effect of vibration is more marked where angular coarse aggregates are used than where aggregates having rounded surfaces are used.

5. In general, it appears from the results of these tests that the proper use of the vibratory method of finishing should result in an improvement of the quality of concrete pavements.

BASE MIX ESTABLISHED FOR STANDARD OF COMPARISON

For each combination of fine and coarse aggregate, a so-called "base mix" was established. This base mix was designed so that the combinations containing the coarse sand would have a cement factor of approximately 6 sacks per cubic yard. For the corresponding mixes with fine sand the proportion of sand was reduced so as to compensate for the differences in grading without changing the water-cement ratio or the proportion of coarse aggregate. Preliminary laboratory tests indicated a reduction in sand content of one tenth part in the case of gravel and two tenths part in the case of stone and slag. This procedure resulted in the use of slightly higher cement factors for the fine sand mixes, the maximum difference being 0.3 sack in the case of the stone concrete. Values of b/b_o, dry-rodded, ranged from 0.73 to 0.78, depending upon the materials being combined. Each of these base mixes was designed to produce concrete with a slump of about 2 inches and was used in a section constructed without vibration as a standard of comparison. For each standard section there were constructed five sections in all of which the concrete was somewhat drier than the standard and all of which were vibrated. In two of the vibrated sections the proportions were the same as the standard or base mix and the drier consistency was obtained by lowering the water content until slumps of approximately one inch and one half inch were obtained. In the other three the water-cement ratio used in the standard or base mix was maintained constant and the drier consistency was secured by adding respectively one fourth, one half, and three fourths parts coarse aggregate by volume to the base proportion. A set of 6 sections, 1 screeded and 5 vibrated, formed 1 day's run. As previously noted, the proportions, slump, water-cement ratio and other data for each section comprising series A are shown in table 2. With the exception of the slag in combination with fine sand, two complete rounds were run, making a total of 66 sections, two with each aggregate combination.

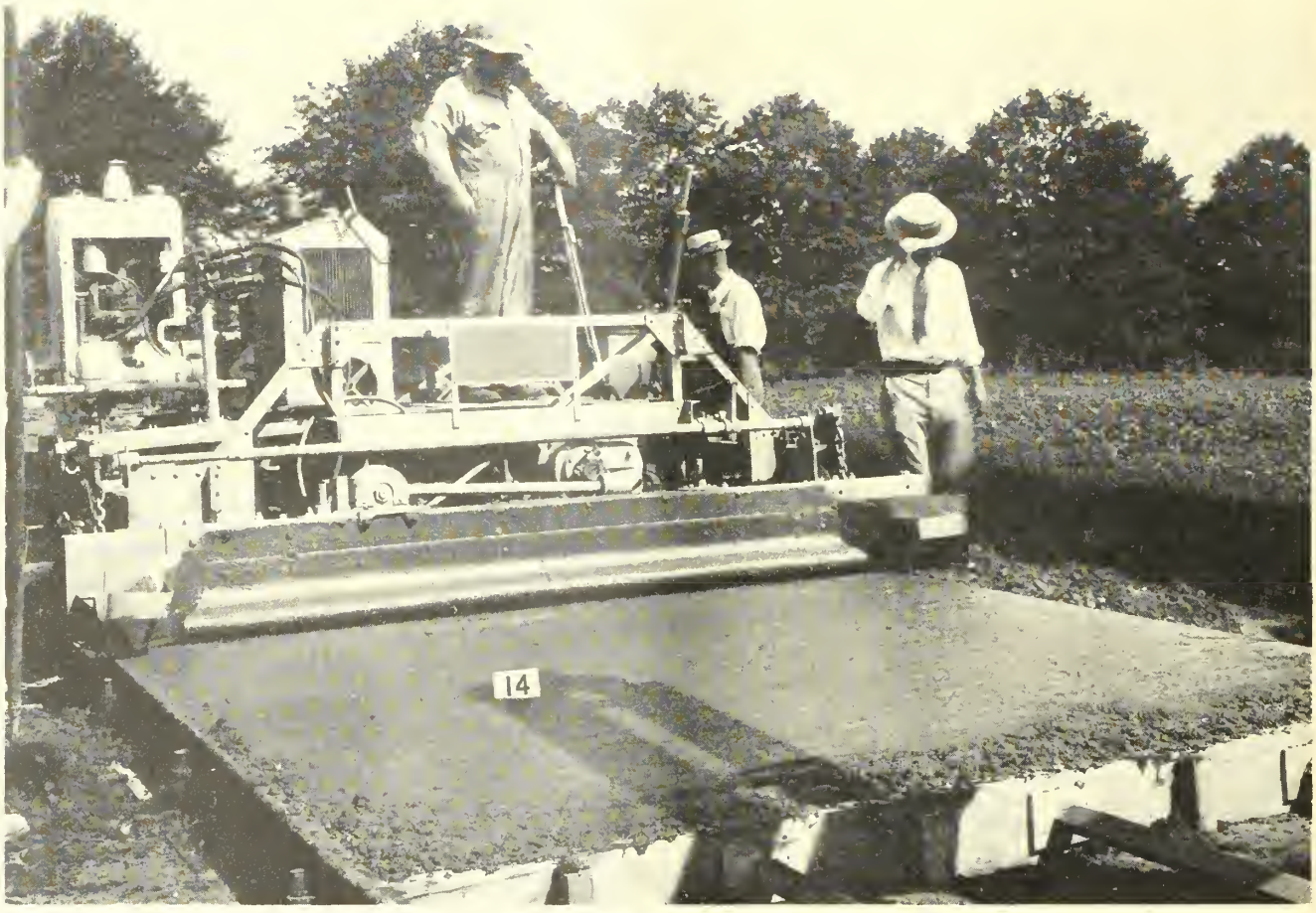


FIGURE 3.—SECTION 14 AFTER FIRST PASSAGE OF FINISHING MACHINE, SHOWING "BULL-NOSE" ATTACHED TO FRONT SCREED.

RESULTS OF TESTS DISCUSSED IN DETAIL

The data are discussed first with the idea of showing in a broad general way the effect of vibration on the average of all comparable concretes containing the various aggregate combinations as compared to the base mix which was placed as the standard of comparison. Thus, on the basis of proportions as shown in table 2, the results of tests on sections 2, 8, 14, 20, 38, 43, 50, and 56 have been averaged and are compared to a similar average of the eight corresponding base sections (sections 1, 7, 13, 19, 37, etc.) to give a general picture of the effect of reducing the slump to about 1 inch without changing the proportions. In a similar way, the average results on sections 3, 9, 15, 21, etc., are compared to the base mix, and so on. It will be noted that each average value for slab strength obtained as above noted includes the results of two tests on each of four aggregate combinations; that is, gravel with coarse sand, gravel with fine sand, crushed stone with coarse sand, and crushed stone with fine sand. Each value therefore represents the average of 32 individual tests. It is felt that these four combinations represent fairly well the typical aggregates in use at the present time. The results for slag are not included in this grouping, first, because slag is not a typical aggregate from the standpoint of distribution and, second, because in the second round slag was not tested in combination with fine sand so that an average including slag could not be obtained from a series containing an equal number of tests of each combination.

The discussion of the data from the above standpoint is followed by a comparison of the effect of variations in material type combinations; that is, the effect of vibration on gravel concrete versus crushed stone concrete, etc. This, in turn, is followed by a detailed discussion of uniformity as revealed by a comparison of the strength and percentage of honeycomb in each of the four test slabs comprising each test section. It is felt that a study of the data from this standpoint is equally as important as a discussion of average results because of the fact that uniformity of quality may be considered as important a characteristic as average quality.

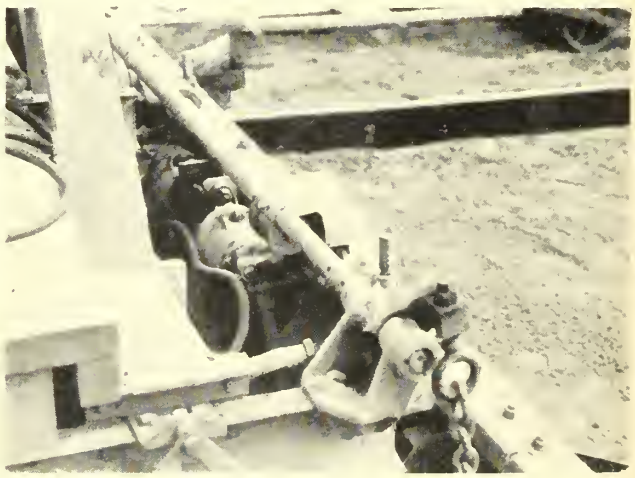
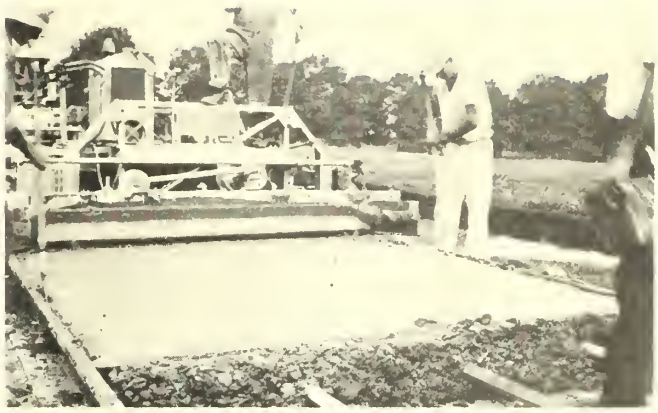


FIGURE 4.—SHOWING ELECTRIC VIBRATING MOTOR MOUNTED ON FRONT SCREED.

Before Finishing.



After First Passage of Finishing Machine.

Figure 5.—Section 23, Vibrated. Aggregates, Fine Sand and Crushed Stone; Proportions, 1:1.7:3.8; Slump, $1\frac{1}{4}$ Inches.

AVERAGE RESULTS SHOW EFFECT OF VIBRATION FAVORABLE WITH 1-INCH SLUMP; ½-INCH SLUMP TOO DRY

The average values for the test results calculated as indicated above are shown in figure 6. In this graph and in those which follow lines have been drawn between the plotted values of crushing strength, modulus of rupture, honeycomb, and strength variation in order to show the trends in the values of these quantities with the indicated variations in slump, water-cement ratio, proportions, and type of finish. These curves should not be looked upon as indicating definite quantitative relations, since the abscissa in no case represents a scalar quantity.

An inspection of this chart reveals a number of interesting general relations. For instance, it will be noted that the vibrated slabs containing the base mix at 1-inch slump (values shown in the second column of figure 6) show somewhat higher strengths and somewhat less honeycomb than the standard-finish base mix at 2-inch slump (column 1). In view of our experience in the first series, where the use of a consistency less than 2-inch slump when finished in the usual way resulted in lower slab strength and increased honeycomb, it seems reasonable to assume that the improvement in quality is due to the method of finishing; that is, the use of vibration. However, when we come to the vibrated base mix at ½-inch slump (column 3) we find a decrease in slab strength as well as a marked increase in honeycomb and less uniformity in strength. This would indicate obviously that the ½-inch slump is too dry for satisfactory placement, even with the use of vibrators as operated in these tests. The only increase in strength for this mix is noted in connection with the crushing strength of cores drilled from the sections. However, this lack of concordance between flexural and crushing strength is not surprising for the following reason: As noted previously, considerable care was taken in drilling cores to select portions of the slabs free from honeycomb. Since a badly honeycombed core would not be tested in ordinary work it was deemed best to confine these tests to sound, homogeneous concrete insofar as possible and to study the effect of honeycomb through a study of variations in flexural strength as well as by direct measurement of the amount of honeycomb in each slab. With the exception of the sections in group 3 ($\frac{1}{2}$-inch slump) the spread between core and cylinder strength is just about as great for the standard-finish sections as for the vibrated sections. This would indicate that, except in the case of the very dry mixes, the relatively higher core strengths are not

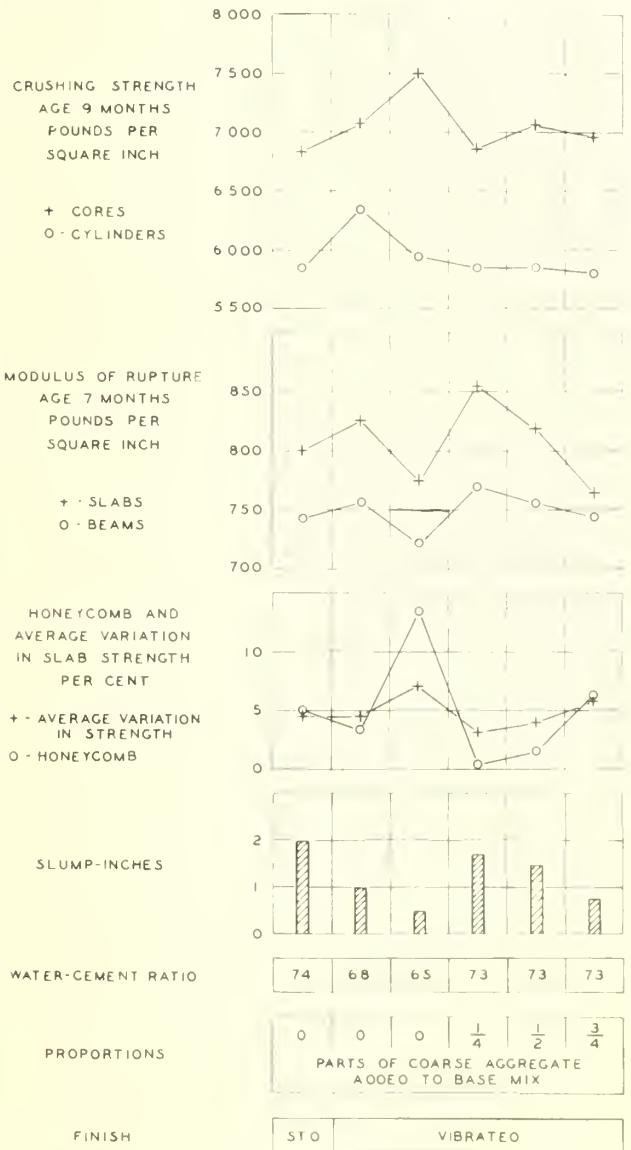


Figure 6.—Series A, Effect of Vibration; Averages for Sections Containing Gravel and Crushed Stone.

due to the effect of vibration. In the case of the concrete having a ½-inch slump it is probable that vibration did tend to increase the crushing strength of the con-

crete. On the other hand, the cylinders showed a decrease due to the fact that the ½-inch slump concrete was probably outside the range of consistency to which the laboratory relation between water-cement ratio and strength applies.

ADDITION OF ONE-FOURTH PART COARSE AGGREGATE INCREASES FLEXURAL STRENGTH AND UNIFORMITY OF VIBRATED SECTIONS

Let us examine next the three groups of sections in which the slump was decreased by increasing the proportion of coarse aggregate instead of reducing the water-cement ratio (columns 4, 5, and 6). It will be observed that the addition of coarse aggregate does not affect the crushing strength of the concrete, but that the flexural strength of the vibrated slabs is increased by the addition of one-fourth part coarse aggregate and is reduced as the amount of coarse aggregate is increased beyond one-fourth part. These trends seem entirely reasonable when we consider that the core strengths are not affected to the same extent as flexural strength by variations in the uniformity of the concrete in the slab. It is interesting to note that, from the standpoint of average flexural strength, as well as from that of uniformity in strength, the sections in group 4, containing one-fourth part additional coarse aggregate, show the best results. It is true that variations in neither strength nor uniformity are of considerable magnitude, the maximum difference in slab strength (column 4 as compared to column 6) being only about 12 percent of the strength of the base mix. The various trends, however, are so consistent and appear so reasonable from a theoretical point of view that they may be assumed to be indicative of real variations in quality and so may be used as the basis for conclusions as to the effect of finishing on the results.

AGGREGATE COMBINATIONS STUDIED SEPARATELY

The next step will be to examine the various material combinations individually in order to ascertain to what extent variations in materials may affect the results. The values for each of the six combinations from which the general averages were obtained have been regrouped in figures 7, 8, and 10, so as to bring out these relationships. The results obtained with gravel as coarse aggregate are plotted in figure 7, with similar data for crushed stone in figure 8. It will be observed that for similar mixes and methods of finishing the sections containing crushed stone, as coarse aggregate show somewhat higher crushing strengths and considerably higher flexural strengths than the sections containing gravel. Attention should be called to the fact that these differences are due to variations in the physical properties of

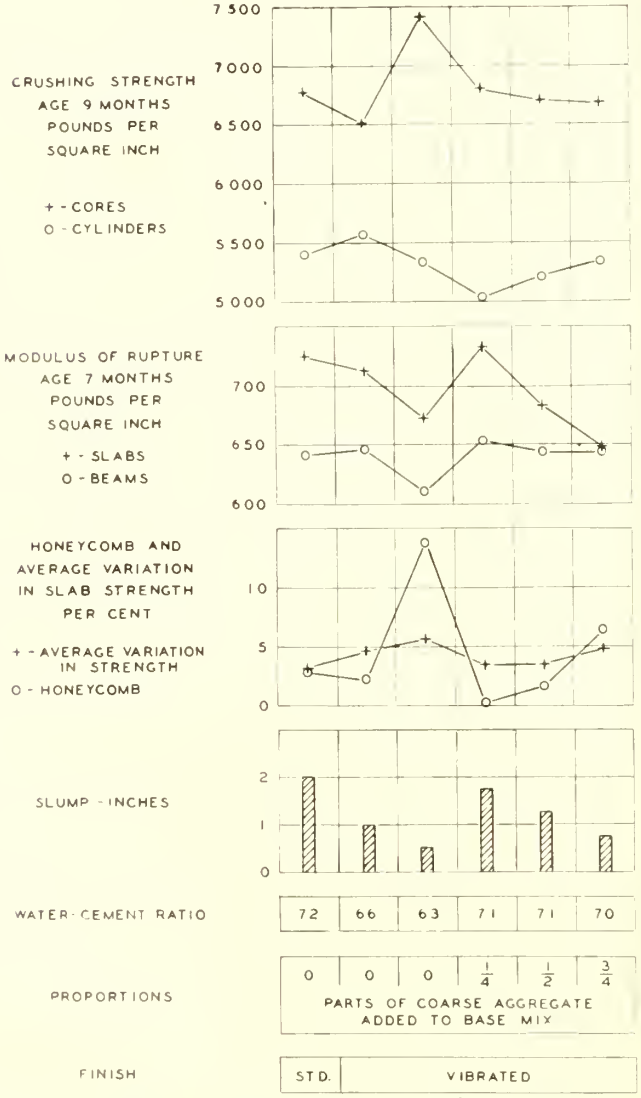


FIGURE 7.—SERIES A, EFFECT OF VIBRATION; AVERAGES FOR SECTIONS CONTAINING GRAVEL.

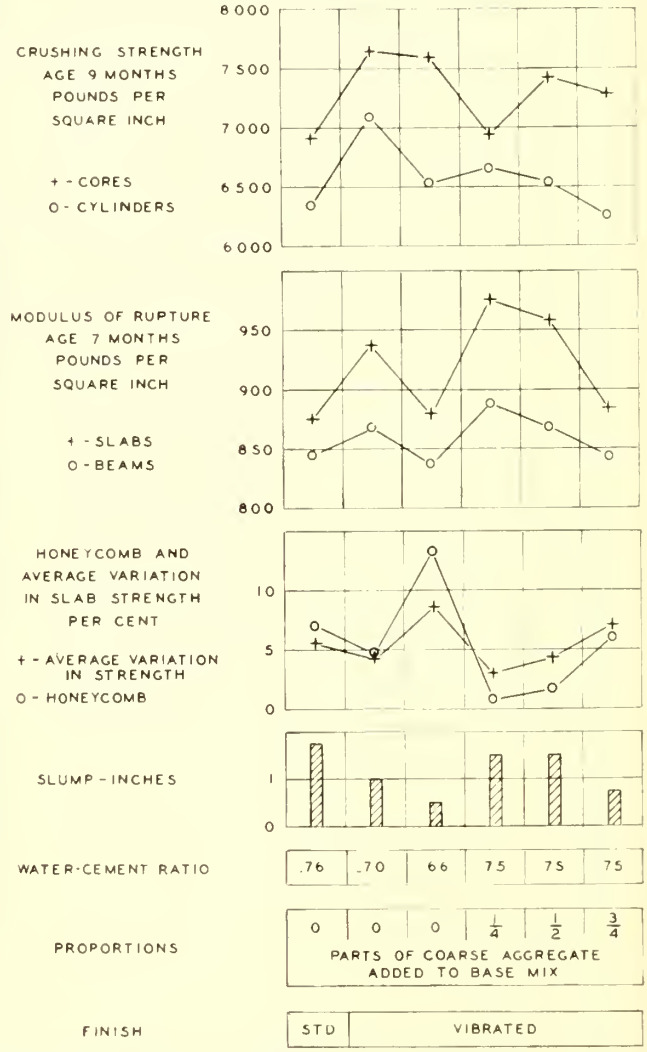


FIGURE 8.—SERIES A, EFFECT OF VIBRATION; AVERAGES FOR SECTIONS CONTAINING CRUSHED STONE.

the particular aggregates used in these tests and not merely to the fact that one aggregate happens to be crushed stone and the other gravel. Aggregates from other sources might well have shown entirely different relative strengths both in compression and in flexure. In figure 10 the average results of tests on concrete containing gravel and crushed stone in combination with coarse sand are compared to the corresponding averages for concrete containing fine sand. The results obtained with slag, which were not included in the general average are shown in figure 9.

An inspection of each chart reveals, in general, the same trends as are shown in figure 6. In all cases the highest slab strength as well as the least honeycomb is found in the vibrated group containing one fourth part additional coarse aggregate (column 4 in each figure). Likewise, increasing the quantity of coarse aggregate beyond one fourth part in all cases decreased the slab strength as well as the uniformity of the concrete as compared with results obtained with one fourth additional part. The curves showing these relations are remarkably consistent for all three coarse aggregate types. It will be noted also that in the sections containing crushed stone and slag even the concrete containing the largest amount of coarse aggregate (column 6 of figs. 8 and 9) is higher in flexural strength than the base mix unvibrated. Uniformity, also, is just about as good.

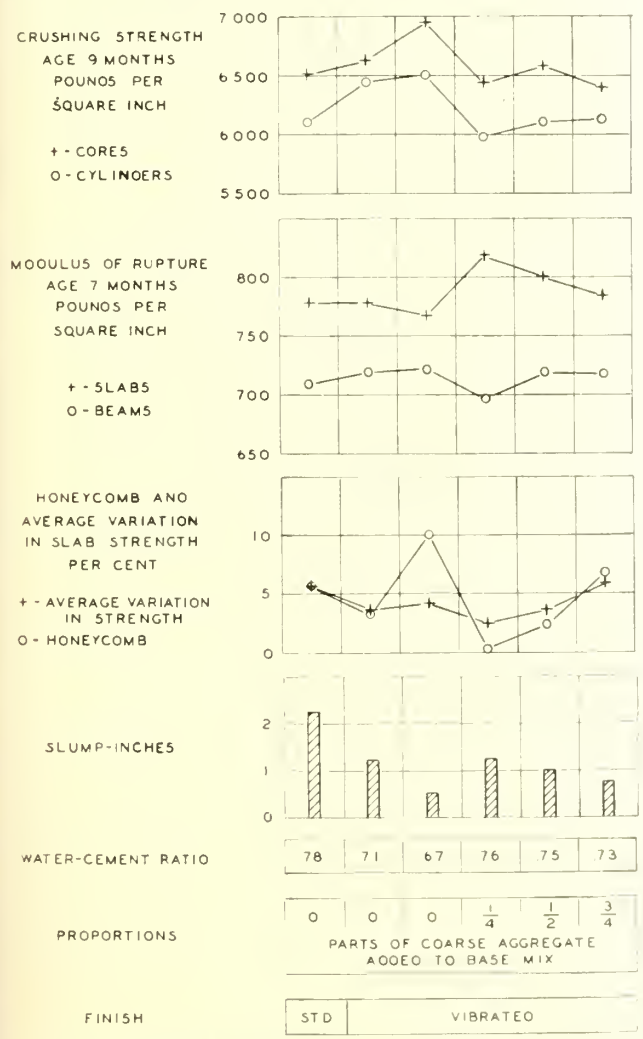


FIGURE 9.—SERIES A, EFFECT OF VIBRATION; AVERAGES FOR SECTIONS CONTAINING SLAG.

The effect of vibration on slab strength is most marked in the case of crushed stone (fig. 8) and least in the case of gravel (fig. 7). This may be observed by comparing the slab strength of the unvibrated concrete (column 1 in each case) with the strengths of the corresponding vibrated sections. In the case of crushed stone all but the very dry concretes (columns 3 and 6) show marked increase in slab strength under vibration. In the case of gravel, group 4 containing one fourth part additional coarse aggregate, is the only one showing higher slab strength than the base or unvibrated concrete. In the case of slag, the three sections carrying additional coarse aggregate are higher in strength than the unvibrated base mix. Both the crushed stone and slag concretes containing the drier mixes (columns 3 and 6 of figs. 8 and 9) show slab strength about equal to the unvibrated concrete in spite of the fact that group 3 in each case has considerably more honeycomb than the base mix. It is interesting to note that, in general, the uniformity of the slag concrete is as good as that shown for crushed stone and gravel.

The inability of the vibrators to finish very dry concrete (½-inch slump) is shown by the relatively low strengths and high percentages of honeycomb in the slabs comprising the third group in each case (column 3 of figs. 7, 8, and 9). Variation in consistency apparently had a smaller effect on the slab strength of slag concrete than either of the other coarse aggregates. Somewhat less honeycomb in the case of the dry mix (group 3) may also be noted. The marked difference in crushing strength of gravel concrete cylinders with respect to cores as compared to similar results with crushed stone and slag should also be noted. In the former case the cores average about 1,500 pounds per square inch higher than the cylinders, which is a considerably greater difference than is shown for either the stone or slag concrete.

COARSE VERSUS FINE SAND

In figure 10, the test results for all sections using crushed stone and gravel in combination with coarse sand as fine aggregate have been averaged and are compared to the results for the corresponding concretes in which the fine sand was used.

The same general relationships appear except that the adverse effect of using the dry mix is more marked in the case of coarse sand. The average slump is, however, considerably lower; that is, ⅛ inch as compared to ½ inch for the fine sand. The average difference in slab strength as compared to beam strength is more marked in the case of the fine sand, especially for the unvibrated concrete and for the two groups of vibrated concrete containing the base mix with lower water-cement ratio. The tendency for the slab strengths to drop at a greater rate than the beam strengths for increasing percentages of coarse aggregate should also be noted in this as well as in the other graphs. The general agreement between the results of strength tests on cores, cylinders, and beams for concretes having practically the same water-cement ratio (columns 1, 4, 5, and 6) is also of interest.

UNIFORMITY OF INDIVIDUAL SECTIONS COMPARED

In order to bring out clearly the effect of honeycomb on slab strength the results of the tests for flexural strength of each of the four 27-inch slabs in each test section have been plotted in figures 12, 13, and 14. The data are also shown in table 5. These are the

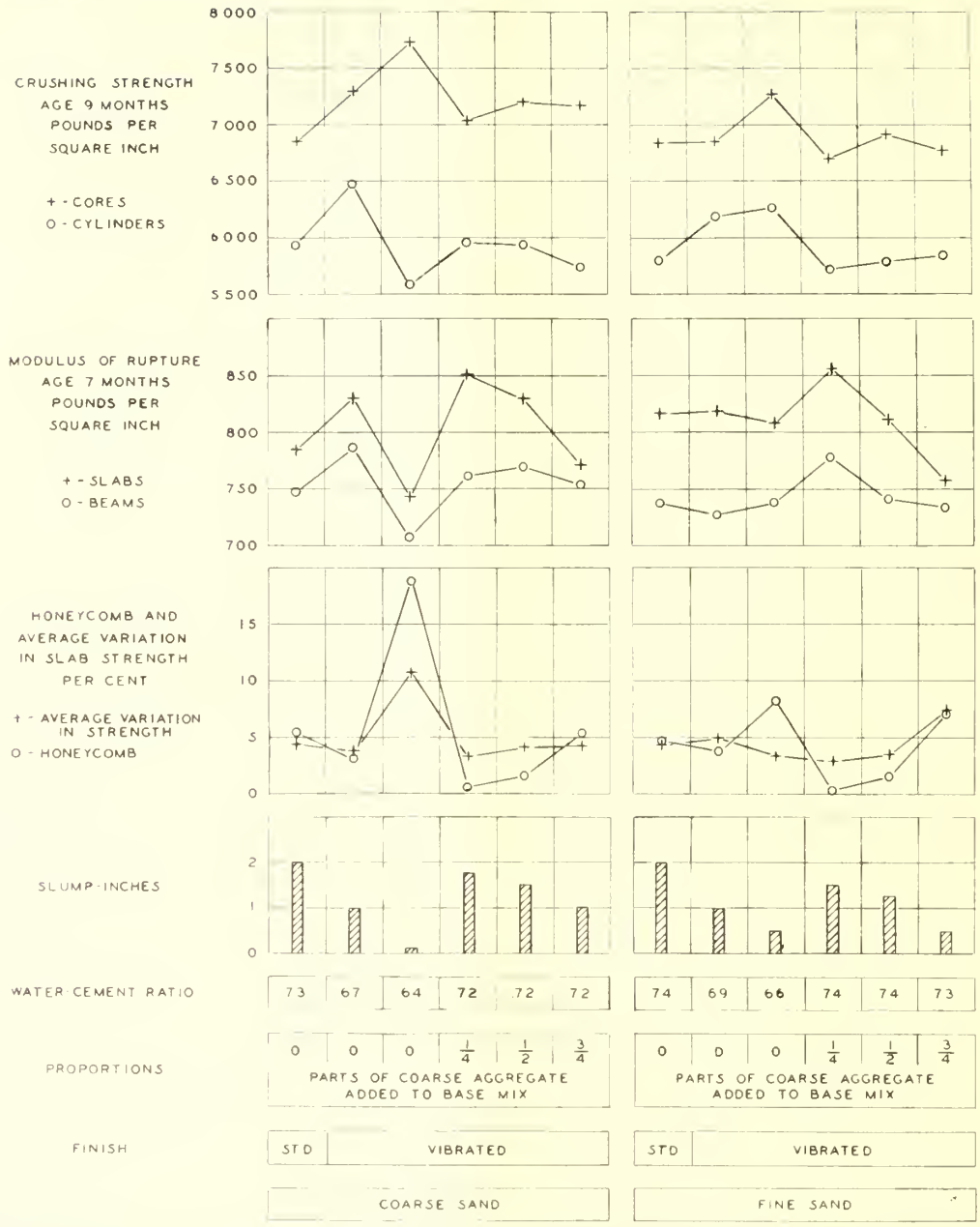
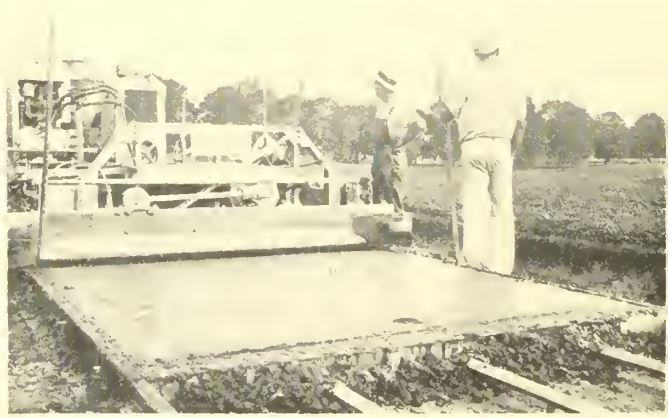


FIGURE 10.—SERIES A. EFFECT OF VIBRATION; AVERAGES FOR SECTIONS CONTAINING GRAVEL AND CRUSHED STONE, SEPARATE ACCORDING TO FINE AGGREGATES USED.

BEFORE FINISHING.



AFTER FINISHING.

FIGURE 11.—SECTION 31, VIBRATED. AGGREGATES, FINE SAND AND SLAG; PROPORTIONS, 1:1.9:3.2; SLUMP, 1 INCH.

individual values from which the average values shown in figures 6, 7, 8, 9, and 10 were derived. The effect of honeycombing on strength is immediately apparent. In practically every case we find that the highest strengths obtained for any test section were on slabs containing less than 3 percent honeycomb, whereas those slabs showing an excess of 10 percent honeycomb developed much lower strengths. The effect of using dry consistencies on the extent of honeycomb is also brought out. Marked examples of the effect of honeycomb on strength and the effect of consistency on the amount of honeycombing are sections 3, 12, and 39 in figure 12 (all of which showed ¼ inch or less slump), sections 15, 21, 24, and 51 of figure 13 (all ¾-inch slump or less) and section 27 in figure 14. On the other hand, those sections showing the least honeycomb and greatest uniformity are, in general, the vibrated sections in which the slump was in excess of 1 inch. There are some exceptions to this, as for instance, section 45 (fig. 12) with 1-inch slump and less than 3 percent honeycomb in any slab. This section also showed a high degree of uniformity. It will be noted that honeycombing exists in almost all of the standard-finish sections for an average slump of 2¼ inches or less. Section 44 (fig. 12) with 2½-inch slump, and section 61 (fig. 14) with 2¾-inch slump, were the only ones with all slabs showing less than 3 percent honeycomb.

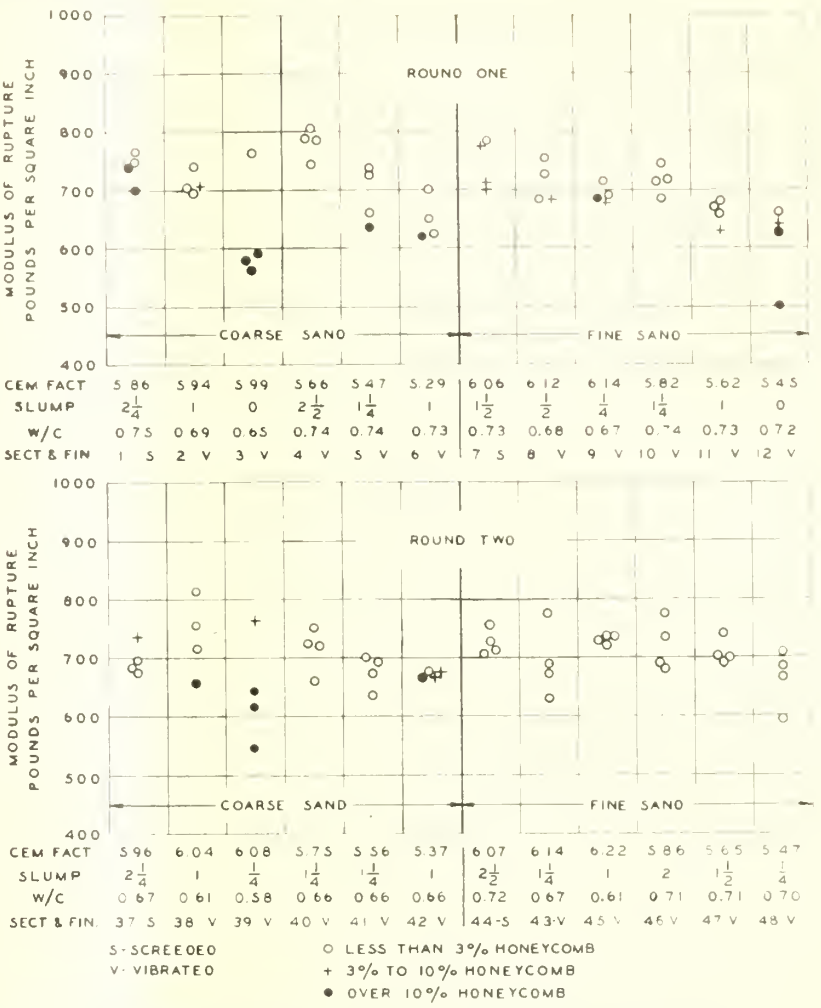

MODULUS OF RUPTURE POUNDS PER SQUARE INCH
ROUND ONE
COARSE SAND
FINE SAND
CEM FACT 5.86 5.94 5.99 5.66 5.47 5.29 6.06 6.12 6.14 5.82 5.62 5.45
SLUMP 2¼ 1 0 2½ 1¼ 1 1½ ½ ¼ 1¼ 1 0
W/C 0.75 0.69 0.65 0.74 0.74 0.73 0.73 0.68 0.67 0.74 0.73 0.72
SECT & FIN 1 S 2 V 3 V 4 V 5 V 6 V 7 S 8 V 9 V 10 V 11 V 12 V
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
ROUND TWO
COARSE SAND
FINE SAND
CEM FACT 5.96 6.04 6.08 5.75 5.56 5.37 6.07 6.14 6.22 5.86 5.65 5.47
SLUMP 2¼ 1 ¼ 1¼ 1¼ 1 2½ 1¼ 1 2 1½ ¼
W/C 0.67 0.61 0.58 0.66 0.66 0.66 0.72 0.67 0.61 0.71 0.71 0.70
SECT & FIN. 37 S 38 V 39 V 40 V 41 V 42 V 44-S 43-V 45 V 46 V 47 V 48 V
S-SCREEDED
V-VIBRATED
○ LESS THAN 3% HONEYCOMB
+ 3% TO 10% HONEYCOMB
● OVER 10% HONEYCOMB

FIGURE 12.—SERIES A, EFFECT OF VIBRATION; FLEXURAL STRENGTH AT 7 MONTHS OF GRAVEL CONCRETE PAVEMENT SLABS.

VARIATION OF CONSISTENCY BETWEEN ROUNDS 1 AND 2 NOTED

Further reference to figures 12, 13, and 14 reveals another interesting fact. It will be noted that, in general, there is considerably less honeycombing in the sections constructed during the second round than in those built during round 1. For instance, 40, or 28 percent, of the 144 individual slabs tested in round 1 showed more than 10 percent honeycomb, whereas only 11, or 9 percent, of 118 slabs tested during the second round showed more than 10 percent. Two groups in the second round were practically free from honeycomb exceeding 3 percent, gravel concrete with fine sand (fig. 12) and slag with coarse sand (fig. 14), the latter group containing only one slab with more than 3 percent honeycomb, whereas the former contained no slabs showing honeycomb in excess of 3 percent. The difference between the first and second round in the case of the fine sand and gravel concrete may be explained by the fact that during the second round the slump was considerably greater than during round 1 although the same proportions were used in both cases. As a matter of fact the average net water-cement ratio during round 2 was somewhat lower than during the first round. This difference in consistency may possibly be explained by the comparatively low relative humidity of the atmosphere (48 percent) on the day on which sections 7 to 12 were laid as compared with the humidity noted during the construction of sections 43 to 48 (65 percent). Both days were sunny and hot, with average temperatures over 90° F. There seems to be no other reason which would account for the marked difference in consistency which was observed.

This difference in consistency illustrates very forcibly the difficulty of controlling this very essential property of concrete under field conditions. Here we have a condition where variations in the humidity undoubtedly affected the consistency to a marked degree. Under such conditions it is impossible to maintain both proportions and water-cement ratio and to expect the consistency to also remain constant. Therefore, inasmuch as consistency must be closely controlled, and it is impractical to be constantly changing batch weights in order to take care of variations therein, the question arises as to whether we should attempt to construct concrete pavements under a straight water-cement ratio specification, rather than a specification in which the proportions and consistency are fixed and the water content is allowed to vary up to a fixed maximum.

In the case of the coarse sand and slag combination (fig. 14) the marked difference in honeycombing observed between the two rounds is not explained by differences in consistency. Only in the case of the unvibrated concrete was the difference in slump more

than one half inch. Sections 27 and 63 show the greatest divergence as far as workability is concerned. The slump in both cases was one half inch. Sections 25 to 30 (round 1) were laid on a damp, cloudy day (average temperature 76° F., relative humidity 88 percent) whereas sections 61 to 66 were laid on a rather cool, clear day with average temperature of 69° F. and relative humidity of 46 percent. Under such conditions it would be reasonable to suppose that the concrete laid during round 1 when the relative humidity was 88 percent would not dry out so fast as the concrete laid during round 2 when the atmosphere was much drier. Actually, the reverse appeared to be the case.

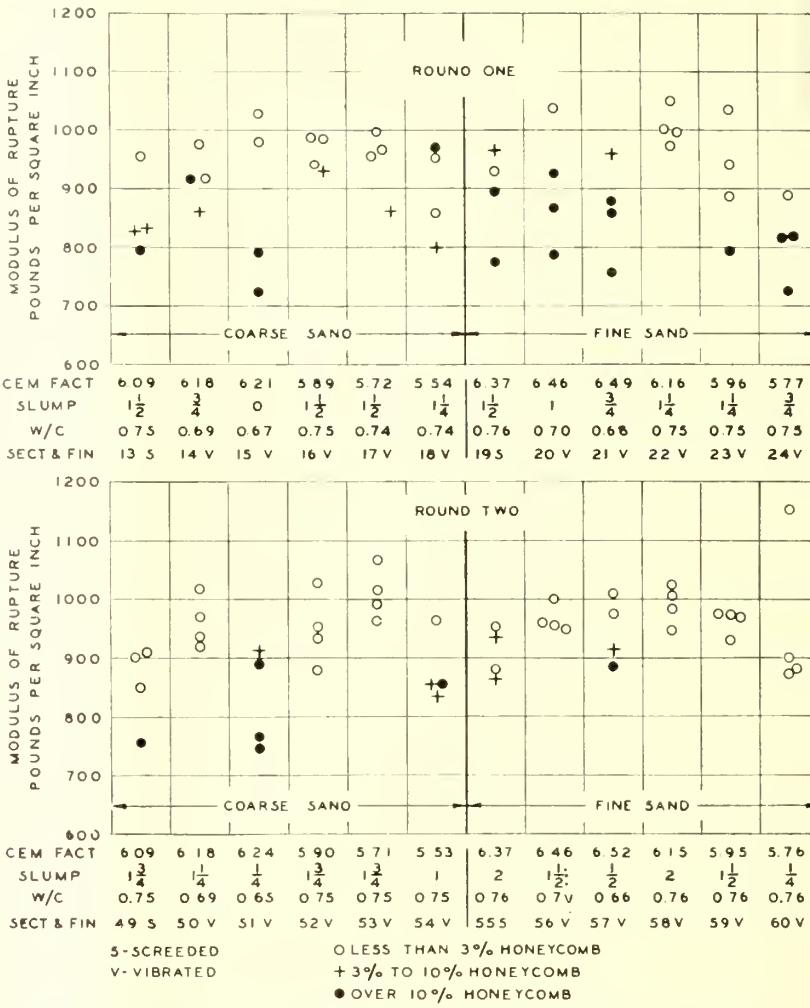


FIGURE 13.—SERIES A, EFFECT OF VIBRATION; FLEXURAL STRENGTH AT MONTHS OF STONE CONCRETE PAVEMENT SLABS.

RELATION BETWEEN WORKABILITY AND SLUMP

To show the average effect of variation in consistency (slump) on the workability of both the vibrated and unvibrated concrete, the average percentage of honeycomb in all slabs constructed of concrete having a given consistency and finished in a given manner has been determined for each value of slump within the range studied, that is, between 0 and 3 inches. The results are shown graphically in figure 17. In this figure, the curve showing the relation between slump and percentage of honeycomb for each method of finishing has been drawn so as to represent a weighted average of the various points. In other words, in establishing the average line, due consideration was given to the number of individual tests represented by each point. This number varied from 1 to 14, depending upon the number of sections having a given slump.

It will be observed in figure 17 that, for a given consistency, the average amount of honeycomb in the slabs finished by vibration is considerably less than in the slabs finished by the standard method. It would appear from these data that the 2-inch minimum slump recommended in the previous report for paving mixes to be finished under standard methods did not, under the conditions of these tests, completely eliminate honeycomb. As a matter of fact, at 2-inch slump an average amount of honeycomb of approximately 5 percent is noted, whereas in the first series honeycombing was almost entirely eliminated at 2-inch slump, except where the so-called "B grading" gravel was used as coarse aggregate. (See fig. 23, PUBLIC ROADS, August 1931.)

Several explanations may be advanced to account for this difference. In the first place the coarse aggregate used in the first series was divided in 3 sizes, whereas only 2 sizes were used in the present series. This may possibly account for the difference in workability. Again in the original series the subgrade was protected with tar paper, whereas, in the present series, the slabs were cast direct upon the subgrade. It is possible that the tar paper, acting somewhat as a form, prevented the segregation of mortar and coarse aggregate on the bottom surface of the slab, thus reducing the percentage of visible honeycomb to a certain extent.

Of course, from the standpoint of this investigation the most important consideration is the difference i slump for standard-finished concrete as compared wit vibrated concrete of the same degree of workabilit (percentage of honeycomb). This difference seems t be in the neighborhood of 1½ inches. For instance, i order to insure concrete substantially free from honey comb (and for the purpose of this discussion 3 percen will be considered the upper limit for this condition) minimum slump of about 2½ inches will be require when the concrete is finished in the usual way compare with about a 1-inch slump when the concrete is vibrated It will be seen therefore that, if anything, the minimu slump of 2 inches recommended in the first repor instead of being higher than necessary was not hig enough to eliminate honeycombing. In other words the results of the present series not only substantiat the conclusions reached in the first report, but seem t indicate that a minimum slump of 2½ inches instead o 2 inches may be advisable in cases where the concret is finished in the usual way.

TENDENCY TO HONEYCOMB IN VALLEY BETWEEN BATCHES SHOWN IN STANDARD SECTIONS

A study of the distribution of honeycomb on th bottom of the four 27-inch slabs composing each tes section of series A reveals some interesting facts. Fo the purpose of this discussion the four slabs in a tes

section will be numbered from 1 to 4, beginning on the right-hand side when facing in the direction of the movement of the paver. Two 27-cubic-foot batches of concrete were required to construct each section. The first batch was dumped from a position in the center of slab 3 and spread both ways into slabs 2 and 4. The second batch was dumped from over the center of slab 1 and spread out into slab 2, overlapping the concrete from the first batch. The longitudinal valley between adjacent batches was therefore in slab 2. It may be noted that of the 11 standard-finished sections in series A, slab 2 showed the greatest amount of honeycomb in 9 cases, with slabs 1 and 4 each showing the maximum in 1 case. This would indicate a distinct tendency to honeycomb in the valley between the two batches and points to the necessity for care in spotting batches of concrete during actual construction in order to minimize this condition.

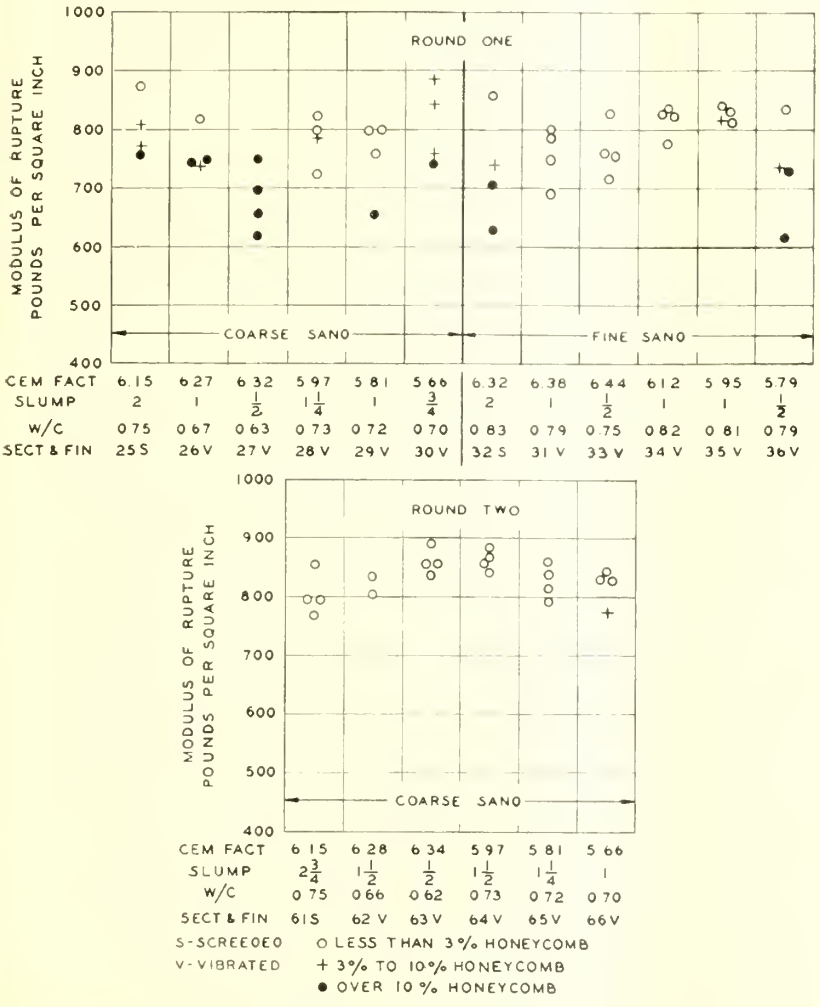

ROUND ONE
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
1000
900
800
700
600
500
400
COARSE SAND
FINE SAND
CEM FACT 6.15 6.27 6.32 5.97 5.81 5.66 6.32 6.38 6.44 6.12 5.95 5.79
SLUMP 2 1 1/2 1 1/4 1 3/4 2 1 1/2 1 1 1/2
W/C 0.75 0.67 0.63 0.73 0.72 0.70 0.83 0.79 0.75 0.82 0.81 0.79
SECT & FIN 25 S 26 V 27 V 28 V 29 V 30 V 32 S 31 V 33 V 34 V 35 V 36 V
ROUND TWO
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
1000
900
800
700
600
500
400
COARSE SAND
CEM FACT 6.15 6.28 6.34 5.97 5.81 5.66
SLUMP 2 3/4 1 1/2 1/2 1 1/2 1 1/4 1
W/C 0.75 0.66 0.62 0.73 0.72 0.70
SECT & FIN 61 S 62 V 63 V 64 V 65 V 66 V
S-SCREEDED
V-VIBRATED
○ LESS THAN 3% HONEYCOMB
+ 3% TO 10% HONEYCOMB
● OVER 10% HONEYCOMB

FIGURE 14.—SERIES A, EFFECT OF VIBRATION; FLEXURAL STRENGTH AT 7 MONTHS OF SLAG CONCRETE PAVEMENT SLABS.

Of the 43 vibrated sections showing honeycomb, slab 2 contained the largest amount in 10 cases, slab 4 in 22 cases, and slab 1 in 11 cases. This distribution shows the greatest amount of honeycomb in the end slabs (nos. 1 and 4) in about 75 percent of the cases and is quite different from that occurring with the screeded sections, where 82 percent of the sections showed maximum honeycomb in slab 2. Inasmuch as the procedure used in dumping was exactly the same, an explanation of the difference may lie in the vibratory effect which was possibly somewhat greater in the center of the screed than at the ends.

NO TENDENCY OF SEPARATORS TO CAUSE HONEYCOMB OBSERVED

At the time of the former investigation, a question was raised as to the effect of the wooden separators installed for the purpose of creating planes of weakness on the amount and distribution of honeycomb in the slabs. It was felt that the separators might have interfered with the distribution of concrete on the subgrade resulting in the concentration of honeycomb areas at and near the separators. A very careful inspection of all of the test slabs comprising this series has failed to reveal any particular trend along this line and it is felt that there was no tendency in this direction. In other words, it seems safe to assume that, other things being equal, the same amount and distribution of honeycomb would have occurred on an actual pavement as was observed in this test.

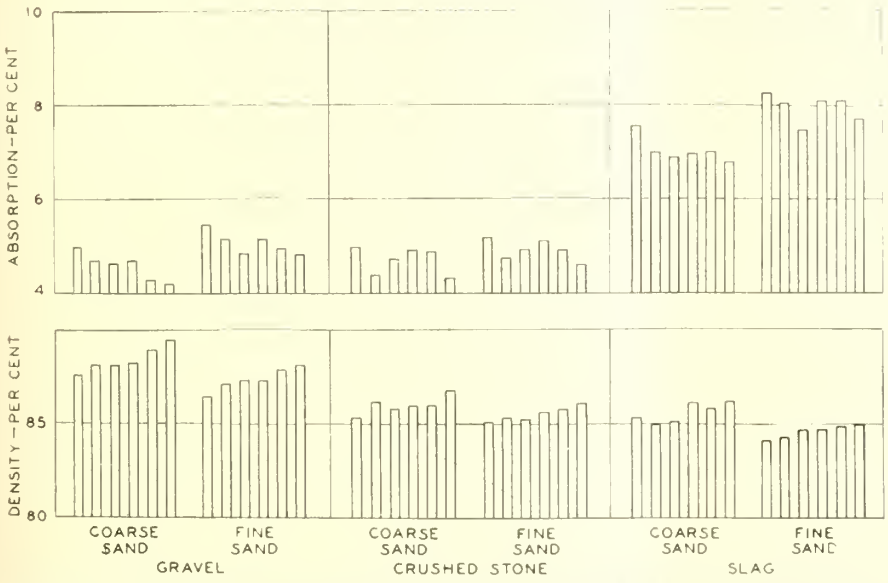
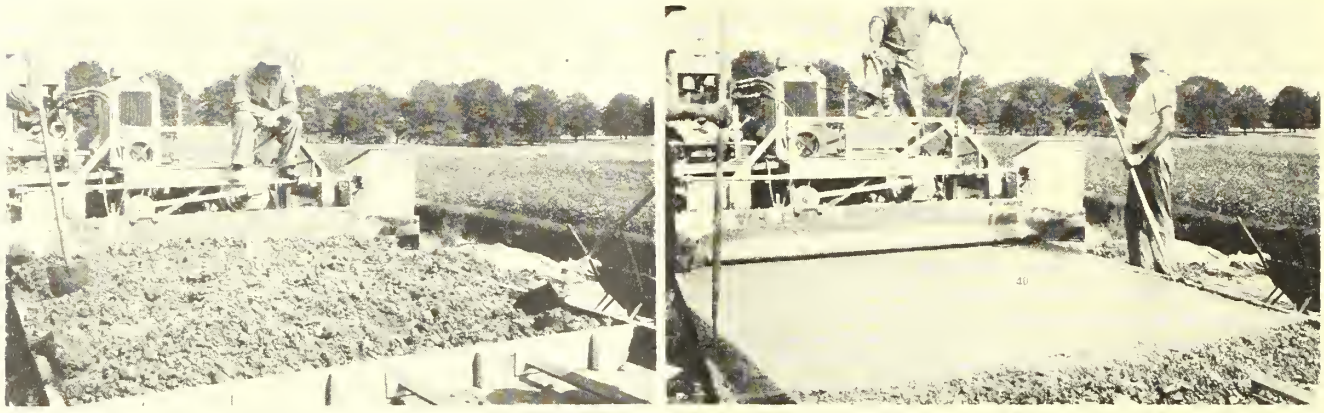


FIGURE 15.—SERIES A, EFFECT OF VIBRATION; RESULTS OF DENSITY AND ABSORPTION TESTS. SECTIONS IN EACH GROUP ARRANGED FROM LEFT TO RIGHT AS IN FIGURE 6.



Before Finishing. *After Finishing.*

Figure 16.—Section 40, Vibrated. Aggregates, Coarse Sand and Gravel; Proportions, 1:1.6:3.7; Slump, 1¼ Inches.

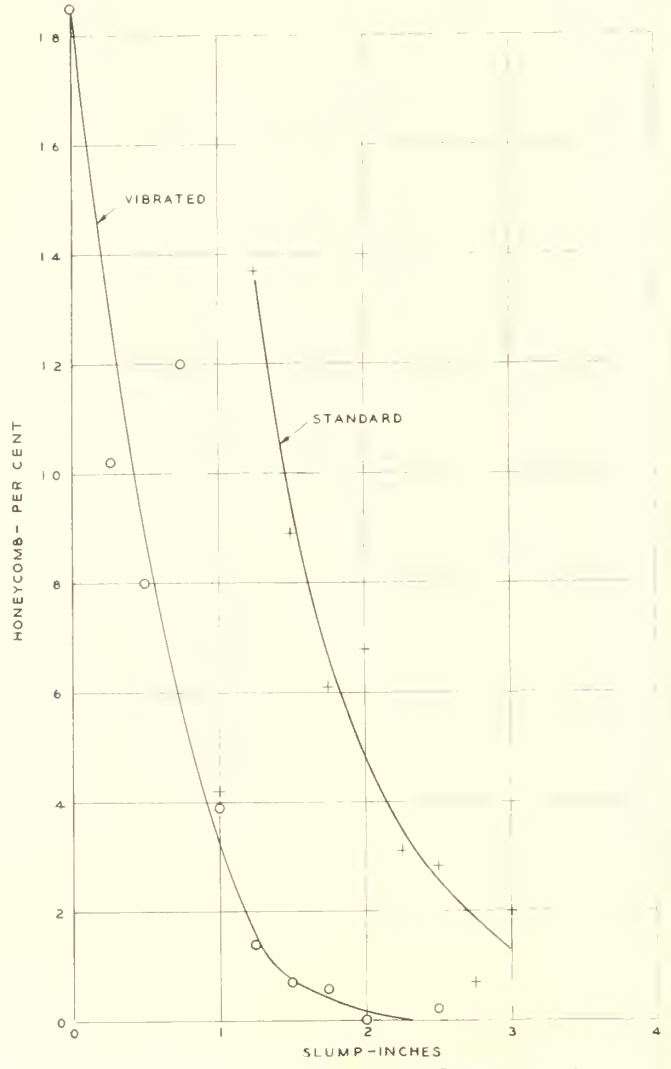


Figure 17.—Series A, Effect of Vibration; Relation Between Honeycomb and Slump.

VIBRATION DOES NOT ADVERSELY AFFECT SURFACE WEAR

The results of the tests for surface wear are shown in figure 18. The depth of wear is reported at the end of both 400 and 600 revolutions. Each value is the average of either 2 or 3 individual tests on different slabs. For each group of 6, the average hardness of the surface of the 5 vibrated sections may be compared directly to the standard-finish section for that particular group. In general, the vibrated sections show slightly less surface wear, although the difference is not sufficiently marked to warrant any general conclusion regarding this property of the concrete. It may safely be assumed, however, on the basis of these tests, that vibration does not adversely affect the surface hardness of the pavement.

It is of interest to note that the hardness of the standard-finish concrete appears to be slightly affected by the material combinations. Sections containing the fine sand seem to be somewhat lower in resistance to wear than the corresponding sections with coarse sand. The same comment applies to the crushed-stone concrete as compared to gravel. In the latter case the difference in hardness is probably due to the fact that a somewhat higher water-cement ratio was used in the crushed stone concrete. Inasmuch as wear was confined to the mortar surface, the type of coarse aggregate did not, of course, directly affect the results.

DENSITY AND ABSORPTION DISCUSSED

In figure 15, there have been plotted average values for density and absorption of cores drilled from the sections in series A. It will be seen that in each group the final densities of the vibrated concretes are, with the exception of groups 2 and 3 in the case of slag with coarse sand, all higher than the densities of the corresponding base mixes; and that, in general, the density increases with increased coarse-aggregate content (nos. 4, 5, and 6 in each group). These variations in density could, of course, be accounted for on the basis of the decreased water or increased proportion of coarse aggregate in the mix provided it could be assumed that the freshly mixed concretes were, in all cases, free from air voids. However, it is felt that in the case of series A where all of the vibrated sections were drier than the base mix and where several carried more coarse aggregate, this assumption cannot be made. In other words, it seems reasonable to assume that in the case of the vibrated sections all of the freshly mixed concrete contained more air voids than the base mix. The fact that the final densities arrange themselves in about the same relative order as the densities of the freshly mixed concretes calculated without correction for air voids is an indication that air voids were eliminated during the finishing operation. Of course, the important feature,

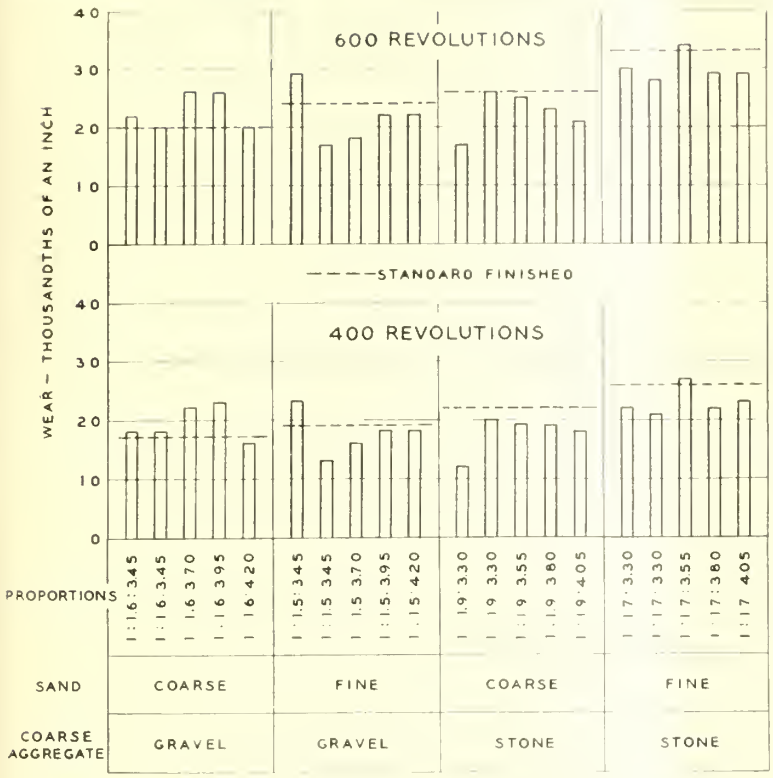


FIGURE 18.—SERIES A. EFFECT OF VIBRATION ON SURFACE WEAR.

so far as these tests are concerned, is that we actually obtained, by the use of the vibratory method of finishing, the increased densities corresponding to the changes in the quantity of water and coarse aggregate which were made. Although density tests were not made during the former tests it is extremely improbable, in view of the inability of the conventional finishing machine to place the harsher mixes properly, that corresponding increases in density were obtained.

Absorption tests, in general, parallel the density tests as would be expected. Absorption is, of course, of interest as a possible measure of the relative ultimate durability of the concrete. The slag concrete, as would also be expected, shows considerably higher absorption than either of the other coarse aggregates.

SERIES B: TESTS TO DETERMINE EFFECT OF DELAYED FINISHING (JOHNSON METHOD)

In this series a method of finishing concrete pavement slabs proposed by Mr. T. H. Johnson, of Sioux City, Iowa, and known as the "Johnson method", was investigated.[1] In all 40 sections were constructed, 16 with gravel as coarse aggregate, 16 with crushed stone as coarse aggregate, and 8 in which the total aggregate was a sand-gravel mixture approximating in grading the so-called "Platte River gravel" used extensively in Nebraska and western Iowa. The gravel and crushed-stone sections were further subdivided as to fine aggregate, an equal number of sections in each group being constructed with coarse and fine sand. Aggregate gradings and other characteristics are given in table 1.

[1] A detailed discussion of the theory underlying this method was presented by Mr. Johnson at the twenty-third annual meeting of the American Concrete Institute in 1927. (See Proc. A.C.I., vol. 23, p. 458.)

JOHNSON METHOD INCREASES STRENGTH AND DENSITY OF CONCRETE

The following conclusions have been derived from tests comparing the Johnson method of finishing pavement concrete with the standard method.

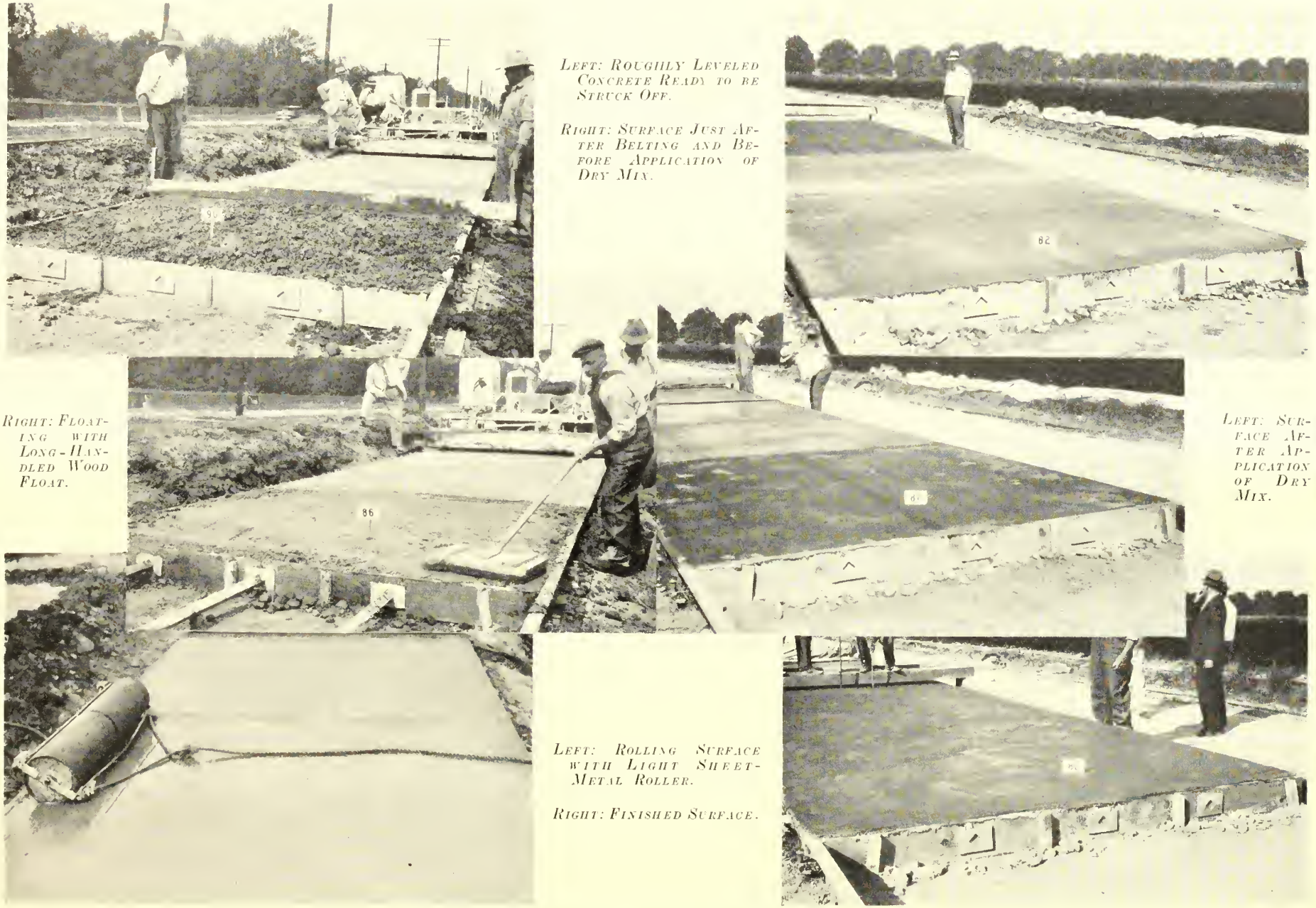
1. The use of the Johnson method of finishing as carried out in these tests will increase the density and strength of concrete pavement slabs by eliminating a larger quantity of excess water before final consolidation than is accomplished by the methods now in common use.

2. The application of the Johnson method of finishing to a mix containing approximately one half part more fine and one part more coarse aggregate than the base mix will produce pavement slabs having substantially the same crushing and flexural strength as the base mix finished by methods now in common use. Such a mix will contain approximately one sack of cement per cubic yard of concrete less than the base mix.

3. In general, the indications of these tests are that the proper use of the Johnson method of finishing should result in an improvement in the quality of concrete pavements.

SAME STANDARD MIX AS IN SERIES A USED AS BASIS OF COMPARISON

For each aggregate combination, except the Platte River gravel, a base mix of the same proportions as used in series A was set up as the standard of comparison. This base mix was finished with the standard double-screed finishing machine in the usual way. For direct comparison with this standard or base mix there were constructed three sections with mixes varying from the base as indicated below. This made a total of four sections per round for each aggregate combination.



Left: Roughly Leveled Concrete Ready to be Struck Off.

Right: Surface Just After Belting and Before Application of Dry Mix.

Right: Floating with Long-Handled Wood Float.

Left: Surface After Application of Dry Mix.

Left: Rolling Surface with Light Sheet-Metal Roller.

Right: Finished Surface.

FIGURE 19.—FINISHING CONCRETE BY THE JOHNSON METHOD.

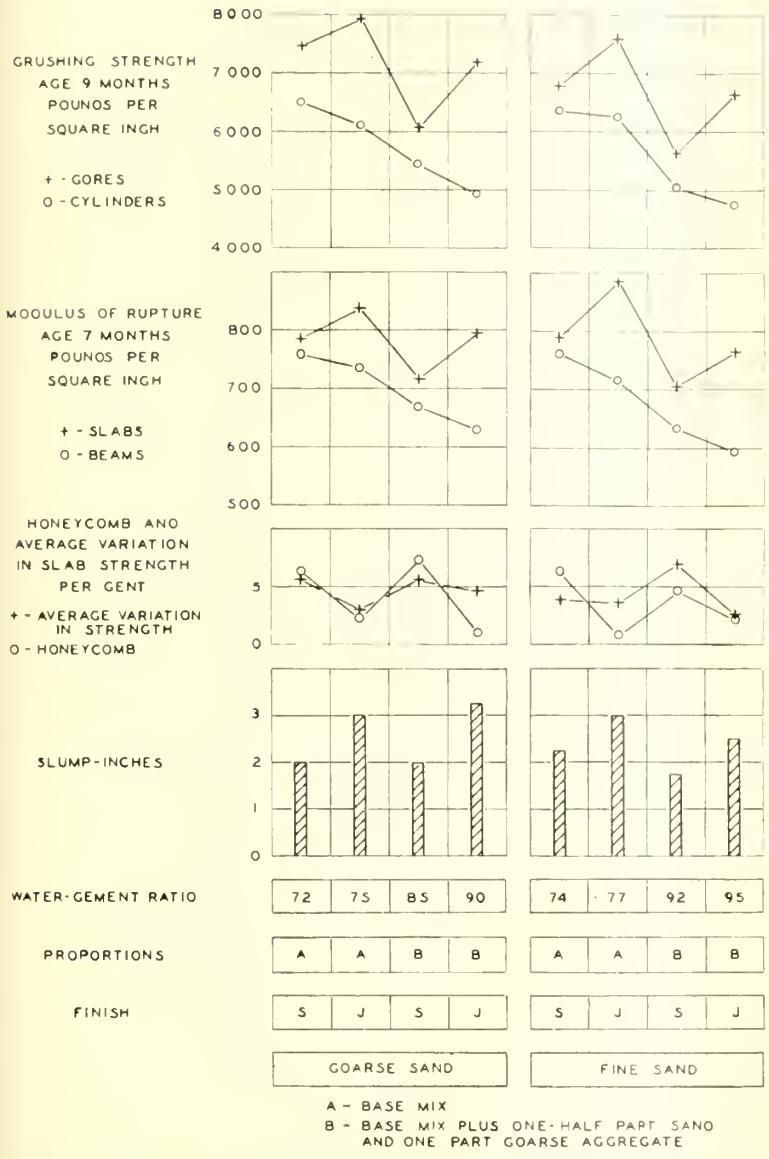


FIGURE 20.—SERIES B, EFFECT OF JOHNSON FINISH; AVERAGES FOR SECTIONS CONTAINING GRAVEL AND CRUSHED STONE.

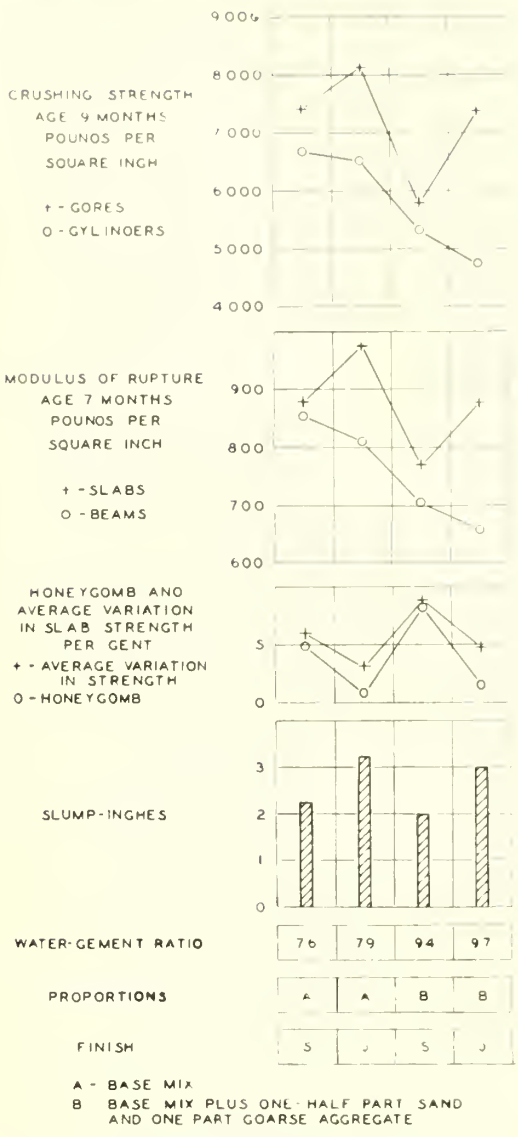


FIGURE 21.—SERIES B, EFFECT OF JOHNSON FINISH; AVERAGES FOR SECTIONS CONTAINING CRUSHED STONE.

For the second section of each group the base proportion was used but the water content was increased to give a slump of approximately 3 inches instead of 2 inches. This section was finished by the Johnson method. The third section in each group was proportioned to contain one half part more fine aggregate and one part more coarse aggregate by volume than the base mix and was finished in the standard manner. The water content in this section was adjusted to give a slump of about 2 inches and the water-cement ratio averaged about 0.15 higher than that of the base mix. The cement factor averaged 1.2 bags per cubic yard less than in the case of the base. The fourth section was a mix similar to the third except that the water content was increased to give approximately 1 inch greater slump. This section was finished by the Johnson method in the same manner as the second section in the group. In table 3 the proportions, slump, water-cement ratio, and method of finishing for each of the sections in series B are given. These data are similar to the data shown in table 2 for series A.

It will be seen that this arrangement makes it possible to compare for each group of four sections the results obtained by the Johnson method with the standard, first, on the basis of the same mix except for a somewhat wetter consistency; and, second, on the basis of not only a wetter but also a considerably leaner mix than the base. It is also possible to compare the lean-mix Johnson-finished sections with a similar mix containing less water and finished in the usual way.

The data for the eight sections in which the sand-gravel mix was used are also shown in table 3. Here the base mix, standard finish, was 1:4 by volume, with sufficient water to give approximately a 2-inch slump. As a comparison, a similar mix with more water was finished by the Johnson method. The group of 4 was completed by 2 sections, 1:4½ mix by volume, 1 finished by the standard method, and 1, somewhat wetter, by the Johnson method. Two complete rounds of tests for each aggregate combination were run in series B, making the total of 40 sections in groups of 4 each as above indicated.

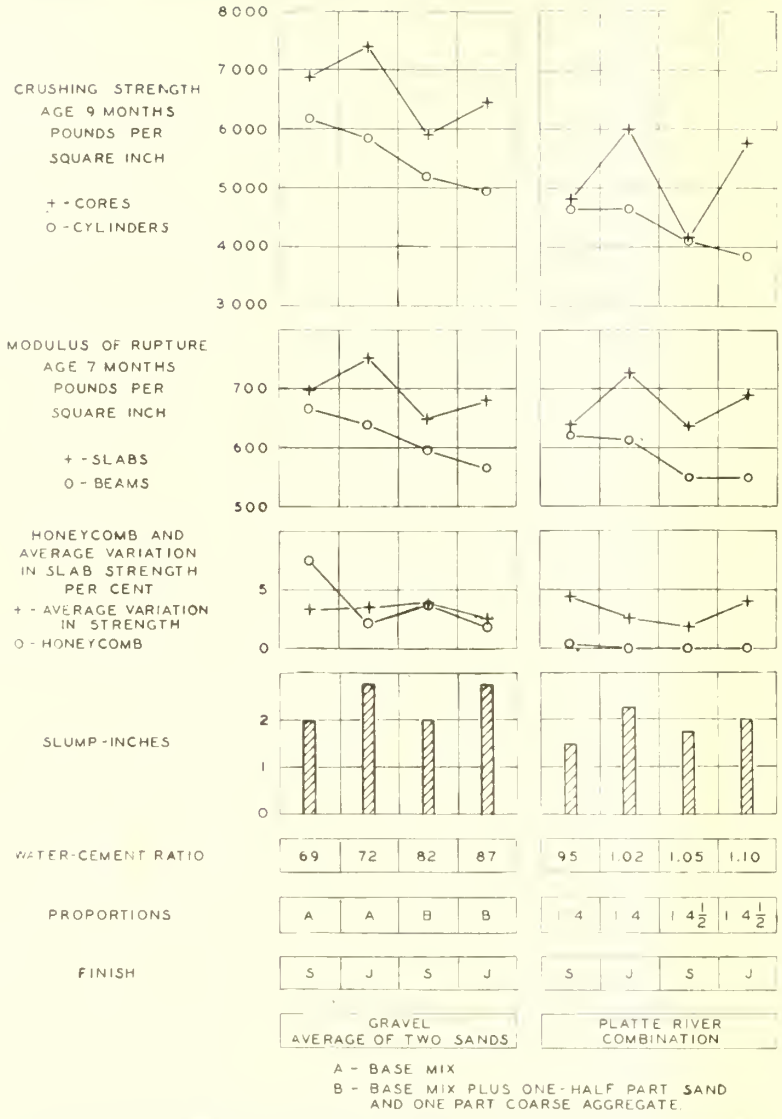


FIGURE 22. SERIES B, EFFECT OF JOHNSON FINISH; AVERAGES FOR SECTIONS CONTAINING GRAVEL AND FOR SECTIONS CONTAINING PLATTE RIVER COMBINATION.

The laying of these sections was started September 29, 1931, and completed October 5, 1931, progress being at the rate of 8 sections per day.

METHODS OF FINISHING DESCRIBED

The procedure used in finishing the concrete by the Johnson method was as follows. Immediately after the concrete was deposited on the subgrade it was struck off with a hand screed. It was then floated with a long-handled wooden float 12 inches wide, 20 inches long, and 2¼ inches thick. The thickness was obtained through the use of a 1¾-inch cypress plank with a ½-inch oak plank nailed to the bottom, the oak making contact with the concrete. After floating, the surface was rolled with a light-weight sheet-metal roller. The roller was operated from side to side of the pavement and was advanced about 1 foot longitudinally with each passage. The entire surface was rolled three times, after which it was belted. This was followed by the application of the so-called "dry mix" to the surface. This mixture was made up of 1 part cement to 1 part of fine sand by volume and was spread over the surface with shovels at a rate of 1 sack of cement per 20 square yards of pavement. For a 7-inch pavement this would be equivalent to increasing the total amount of cement used about 0.25 bag per cubic yard of concrete. Just enough of the mixture was applied to give a very thin continuous cover. (See fig. 19.) The sand used for this purpose was the same as the fine sand used in the concrete mixture and contained approximately 5 percent of moisture when mixed with the cement. After an interval of 11 to 50 minutes, depending upon the amount of water brought to the surface by the dry mix, the surface was again belted.

The second belting was followed by floating with hand floats at intervals until there was little moisture brought to the surface and the concrete became sticky and was drawn along by the float. This general procedure was sometimes varied by belting between floating operations and by additional rollings. The number of floatings necessary to bring the surface to the required condition after the dry mix was applied varied from 3 to 7 and the elapsed time from the placing of the concrete to the final floating varied from 2¼ to 3¾ hours. The various details of the operation as described above are illustrated in figure 19.

COMPARATIVE RESULTS BY JOHNSON AND STANDARD METHODS ANALYZED

The effect of the Johnson method of finishing on the strength and uniformity of the concrete is shown graphically in figures 20 to 25, inclusive. The data are shown in the same manner as the corresponding data for series A, and include average results of crushing-strength tests on cores and cylinders, tests for modulus of rupture on slabs and control beams, and determinations of uniformity, measured by (*a*) the amount of honeycomb in the slabs and (*b*) the average percentage variation in slab strength. In figure 20 the average results of tests for the crushed stone and the gravel concrete for each sand are shown. Figure 21 gives similar data for the crushed stone concrete only, while in figure 22 the corresponding results for the gravel and the Platte River combination are shown. In the case of the crushed stone and gravel concrete (figs. 21 and 22) the values for the sections containing the fine and coarse sand have been averaged. Figures 23, 24, and 25 give, for series B, the results of tests on individual slabs taken from the test sections.

It will be noted that in the case of figures 20, 21, and 22 the results are plotted so as to provide, for each proportion, a direct comparison between the strengths obtained under the Johnson method and those obtained under the standard method. Each point represents the average of tests on four sections, except that in the case of the Platte River concrete (fig. 22, right-hand panel) each point averages only two sections. An inspection of these figures shows at once some very interesting trends.

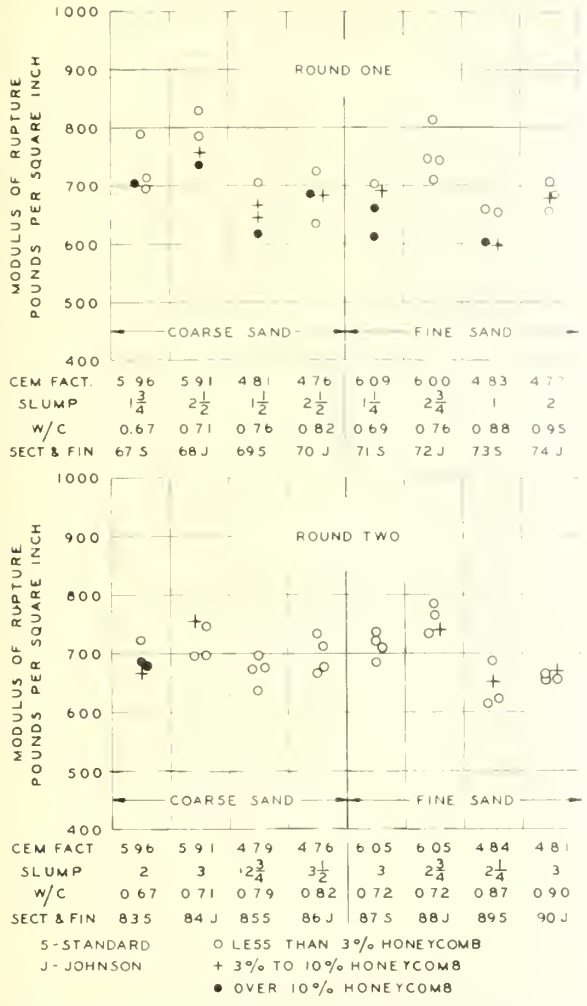

ROUND ONE
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
COARSE SAND
FINE SAND
CEM FACT 5 96 5 91 4 81 4 76 6 09 6 00 4 83 4 77
SLUMP 1¾ 2½ 1½ 2½ 1¼ 2¾ 1 2
W/C 0.67 0 71 0 76 0 82 0 69 0 76 0 88 0 95
SECT & FIN 67 S 68 J 69 S 70 J 71 S 72 J 73 S 74 J
ROUND TWO
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
COARSE SAND
FINE SAND
CEM FACT 5 96 5 91 4 79 4 76 6 05 6 05 4 84 4 81
SLUMP 2 3 2¾ 3½ 3 2¾ 2¼ 3
W/C 0 67 0 71 0 79 0 82 0 72 0 72 0 87 0 90
SECT & FIN 83 S 84 J 85 S 86 J 87 S 88 J 89 S 90 J
S - STANDARD
J - JOHNSON
O LESS THAN 3% HONEYCOMB
+ 3% TO 10% HONEYCOMB
● OVER 10% HONEYCOMB

FIGURE 23.—SERIES B, EFFECT OF JOHNSON FINISH; FLEXURAL STRENGTH AT 7 MONTHS OF GRAVEL CONCRETE PAVEMENT SLABS.

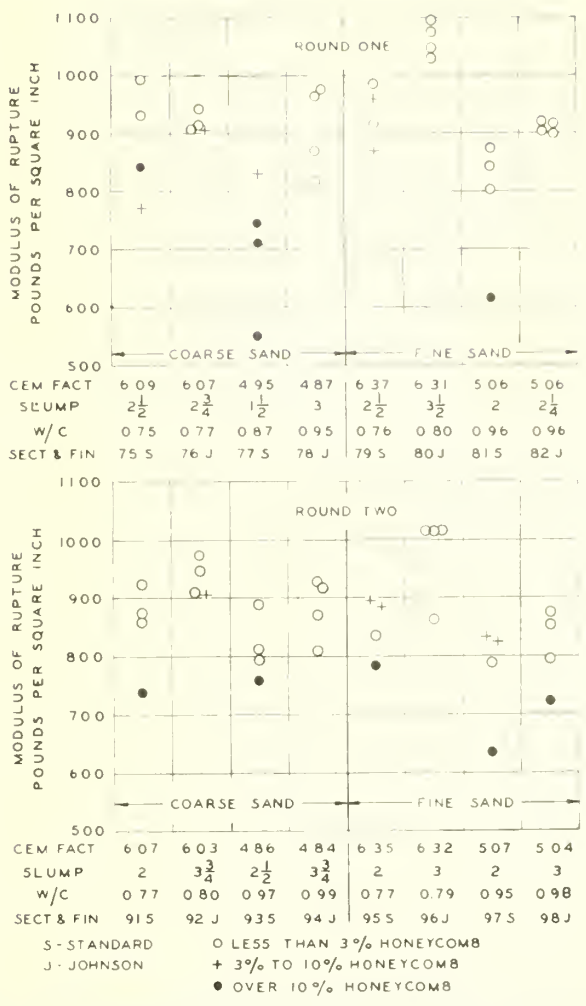

ROUND ONE
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
COARSE SAND
FINE SAND
CEM FACT 6 09 6 07 4 95 4 87 6 37 6 31 5 06 5 06
SLUMP 2½ 2¾ 1½ 3 2½ 3½ 2 2¼
W/C 0 75 0 77 0 87 0 95 0 76 0 80 0 96 0 96
SECT & FIN 75 S 76 J 77 S 78 J 79 S 80 J 81 S 82 J
ROUND TWO
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
COARSE SAND
FINE SAND
CEM FACT 6 07 6 03 4 86 4 84 6 35 6 32 5 07 5 04
SLUMP 2 3¾ 2½ 3¾ 2 3 2 3
W/C 0 77 0 80 0 97 0 99 0 77 0.79 0 95 0 98
SECT & FIN 91 S 92 J 93 S 94 J 95 S 96 J 97 S 98 J
S - STANDARD
J - JOHNSON
O LESS THAN 3% HONEYCOMB
+ 3% TO 10% HONEYCOMB
● OVER 10% HONEYCOMB

FIGURE 24.—SERIES B, EFFECT OF JOHNSON FINISH; FLEXURAL STRENGTH AT 7 MONTHS OF STONE CONCRETE PAVEMENT SLABS.

JOHNSON METHOD SHOWS HIGHER CORE AND SLAB STRENGTHS

For instance, it will be noted that, although a somewhat wetter mix and consequently higher water-cement ratio was used in each Johnson-finished section as compared with the corresponding standard-finished concrete, in every case higher core and slab strengths were obtained on the sections finished by the Johnson method. On the other hand, the strengths of the control specimens representing the concrete going into the Johnson-finished slabs were lower than those representing the corresponding standard sections. This, of course, would be expected because of the higher water ratios. The fact that higher strengths were obtained for the Johnson finish is undoubtedly due, at least insofar as crushing strength is concerned, to the special manipulation which was given the concrete, resulting in the removal of considerable water before final consolidation took place. The control specimens were, of course, not subjected to this manipulation and consequently retained the excess water with a resultant lowering of the strength. As to the increased slab strength shown by the Johnson finished concrete, it is possible that a portion of the increase is due to the influence of consistency on honeycombing and, consequently, on slab strength, entirely apart from the removal of water during the finishing operation.

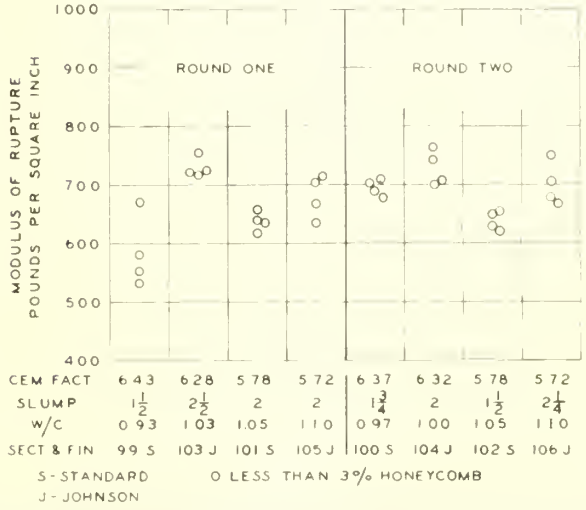

ROUND ONE
ROUND TWO
MODULUS OF RUPTURE POUNDS PER SQUARE INCH
CEM FACT 6 43 6 28 5 78 5 72 6 37 6 32 5 78 5 72
SLUMP 1½ 2½ 2 2 1¾ 2 1½ 2¼
W/C 0 93 1 03 1.05 1 10 0 97 1 00 1 05 1 10
SECT & FIN 99 S 103 J 101 S 105 J 100 S 104 J 102 S 106 J
S - STANDARD
J - JOHNSON
O LESS THAN 3% HONEYCOMB

FIGURE 25.—SERIES B, EFFECT OF JOHNSON FINISH; FLEXURAL STRENGTH AT 7 MONTHS OF PAVEMENT SLABS MADE WITH PLATTE RIVER COMBINATION.

Referring now specifically to figure 20, let us compare first the results of the core tests for the base mix (proportion A). The average crushing strength of sections containing both coarse and fine sand shows an increase for the Johnson sections of about 600 pounds per square inch or approximately 8 percent of the average strength of the standard-finish base mix. The corresponding control cylinders show a slight decrease in strength as would be expected. This decrease averages about 250 pounds per square inch and corresponds to an increase in water-cement ratio of 0.03. For the leaner mix (proportion B) the average increase in core strength is about 1,000 pounds per square inch, or approximately 18 percent. The grand average increase, including all sections, is therefore about 13 percent in favor of the Johnson method of finishing. Inasmuch as the only difference, aside from water content and consistency, was the method of finishing, it may be concluded that the higher average strength shown for the cores drilled from the Johnson sections is due to the method of finishing. It is also interesting to note that the average core strength of the Johnson-finished slabs in the lean mix (ave. $w/c = 0.93$) is only about 200 pounds per square inch lower than that of the corresponding cores for the richer mix (ave. $w/c = 0.73$) with standard finish. The cylinders are about 1,600 pounds per square inch lower in strength.

The same general trends follow in the case of flexure. For proportion A, the average increase in modulus of rupture in favor of the Johnson finish is about 75 pounds per square inch or roughly 9 percent. For proportion B, the increase is about the same. The control beams, which were, of course, fabricated in the usual way show no increase but, on the other hand, a decrease in strength with increase in water content in the same manner as the control cylinders. Considering the small number of specimens represented by each point (8 in the case of cores, cylinders, and beams, and 16 in the case of slabs) the results are remarkably concordant. As in the case of the cores the Johnson-finished slabs containing the lean mix show an average modulus of rupture only very slightly lower (about 10 pounds per square inch) than the standard-finish slabs containing the base mix. On the other hand, the average decrease in the strength of the control beams is approximately 150 pounds per square inch.

It will also be noted that although there was quite an appreciable difference in the grading of the two sands (see table 1) the strengths of the sections containing the coarse sand are not greatly different from corresponding sections containing fine sand, although there is a tendency for somewhat lower strengths in the latter case. The same comment was made in connection with series A. Reference to table 3 shows that in the case of gravel the concretes containing the fine sand contained one tenth part less sand by volume than the corresponding coarse-sand concretes. In the case of crushed stone, the decrease in sand content was 0.2 part. These changes in sand content resulted in an increase in cement content not exceeding in any case 0.30 sack per cubic yard.

HONEYCOMB LESS IN JOHNSON-FINISHED SLABS

With respect to uniformity, figure 20 shows the following facts: The average percentage of honeycomb in all cases is less for the Johnson-finish slabs, the average being about 2 percent as compared to 6 percent for the standard finish. This difference in honeycomb is probably due primarily to differences in consistency. Referring back to figure 17, it will be noted that, under the conditions of these tests, the percentage of honeycomb in the standard-finish slabs was reduced to less than 3 percent only by the use of a minimum slump of 2½ inches. Referring again to figure 20, it will be seen that the average slump of the standard-finish concrete was 2⅛ inches for proportion A and 1⅞ inches for proportion B. The Johnson-finished concrete, on the other hand, showed an average slump of 3 inches in the case of proportion A and 2⅞ inches in the case of proportion B. Knowing the effect of honeycombing on slab strength (see figs. 12, 13, 14, 23, 24, and 25) the question arises as to how much of the increased slab strength shown for the Johnson method was due to the fact that less honeycomb developed than in the standard-finish concrete and how much was due to the elimination of water. This particular point will be discussed in detail in connection with the discussion of individual slab-strength results.

Figure 21 shows data similar to figure 20, except that in this case only the crushed-stone concrete is considered. The results are the average of sections containing the fine and coarse sand. The corresponding values for the gravel concrete are shown in the left-hand panel of figure 22. In connection with the difference in the strengths shown by the crushed stone and the gravel concrete, attention is called to the discussion of figure 6 under series A, in which it was emphasized that this difference is due to the particular aggregates used in this work and does not express a general relation. A comparison of these figures indicates that the increase in strength shown for the Johnson mix is, in general, more marked in the case of the crushed-stone concrete.

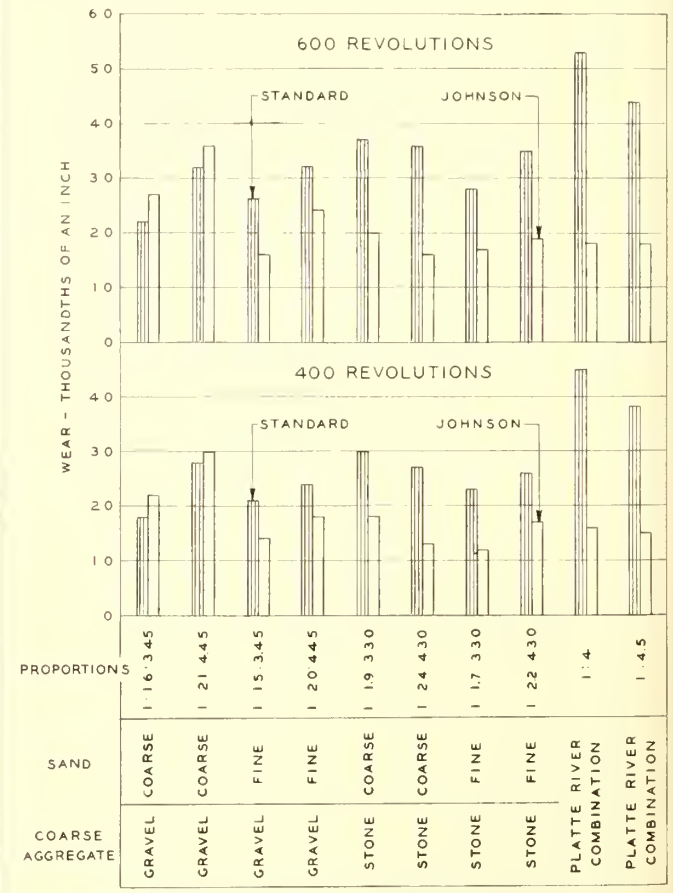


FIGURE 26.—SERIES B, EFFECT OF JOHNSON METHOD OF FINISHING ON SURFACE WEAR.

For instance, for proportion A the increase in core strength is about 700 pounds per square inch for crushed stone as compared to about 500 pounds per square inch for gravel. In the leaner mix the difference is more marked, being about 1,600 pounds per square inch for the crushed-stone concrete as compared to about 550 pounds per square inch for the gravel concrete.

In flexure the differences are of the same order. For proportion A the Johnson mix shows an increase for crushed stone of about 95 pounds per square inch as against approximately 55 pounds per square inch for the gravel concrete. For the leaner mix the difference is about 110 pounds per square inch for crushed-stone concrete as compared to only about 30 pounds per square inch for the gravel concrete. That the large increase shown for the crushed-stone concrete in the lean mix in favor of the Johnson method is due in part to variations in workability is evidenced by the relatively large amount of honeycomb (8 percent) shown for standard-finish sections having this particular combination as compared to only 1½ percent for the corresponding Johnson mix. Here again the question arises as to how much of the difference in slab strength is due to difference in honeycomb and how much to the elimination of water by the Johnson finishing method. This question, however, cannot be raised in the case of the cores because, it will be recalled, special efforts were made to select only sound, uniform cores free from honeycomb. It has already been shown that the crushing strengths of cores taken from the Johnson slabs averaged about 13 percent higher in strength than cores from the corresponding standard-finish sections.

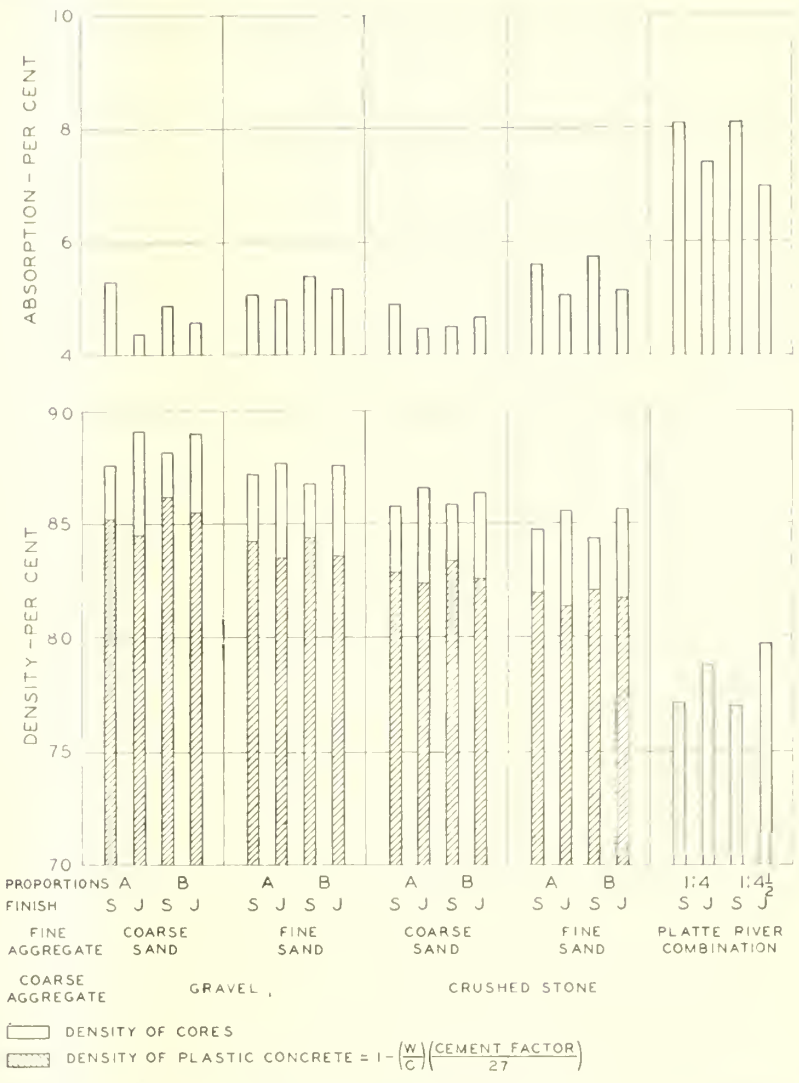


FIGURE 27.—SERIES B, EFFECT OF JOHNSON FINISH; RESULTS OF DENSITY AND ABSORPTION TESTS.

CRUSHING AND FLEXURAL STRENGTH OF CONCRETE MADE WITH PLATTE RIVER COMBINATION INCREASED BY USE OF JOHNSON METHOD

In the right-hand panel of figure 22 there have been plotted the results of tests on the concrete containing aggregates graded similarly to the Platte River gravel. (See table 1.) Two mixes were used, 1:4 and 1:4½ by volume. This aggregate contained practically no material over one half inch in size, and consequently the concrete was not honeycombed in the same sense as the concretes containing normally graded coarse aggregate. Reference to the figure will show the same trends as regards strength as were noted in the case of the gravel and crushed-stone concrete. The increase in crushing strength for the Johnson-finished concrete averages about 1,400 pounds per square inch for the two mixes, with the 1:4½ Johnson-finshed concrete about 1,000 pounds per square inch higher than the 1:4 standard-finish sections. In flexure the same trend is noted. Here the average strength of Johnson-finish slabs is about 70 pounds per square inch higher than the corresponding standard-finish slabs, with the 1:4½ Johnson-finish concrete about 50 pounds per square inch higher than the 1:4 standard finish. Inasmuch as the Platte River "concrete" is, to all intents and purposes, a mortar and not a concrete at all, honeycombing, due to segregation of coarse aggregate is entirely missing. Therefore, the question which may be raised in connection with the standard coarse aggregate concrete as to the effect of honeycombing on strength would not apply in this case. Nevertheless, a very substantial increase in slab strength is noted. This fact, coupled with the undoubted effect of the Johnson method of finishing on crushing strength, leads to the conclusion that, entirely aside from the effect of honeycomb on strength, the flexural strength of concrete is increased by this method of manipulation.

RELATIONS SHOWN BETWEEN CONSISTENCY, HONEYCOMB, AND STRENGTH

In figures, 23, 24, and 25, the relation between consistency, honeycomb, and slab strength for each test slab in series B is shown. It will be of interest to note and comment upon some of these relationships. In the first place it will be seen that the groups of slabs representing the Johnson-finish sections show, in general, somewhat higher strengths and somewhat less honeycomb than the corresponding standard-finish slabs.

The relation between honeycomb and consistency may also be traced. In general, the sections substantially free from honeycomb are those in which the slump is greater than 2½ inches. The sections containing the drier mixes (2-inches slump or less) are, in general, the ones which contain the most honeycomb. There are some exceptions to this broad classification, principally in the range in consistency between 2 inches and 2½ inches. The adverse effect of honeycomb on the strength of individual slabs is very marked in the case of the crushed stone concrete (see fig. 24).

It will be of interest to compare the flexural strength of the standard-finished and Johnson-finished sections on the basis of those slabs which show less than 3 percent honeycomb (the open circles of figs. 23 and 24). A comparison of strength in this manner should eliminate to a large degree the influence of honeycomb on strength. On this basis it is possible to make 15 direct comparisons between the two methods of finishing. Sections 77-S and 78-J cannot, of course, be compared, because of the fact that all four slabs in section 77-S show more than 3 percent honeycomb.

Referring now to figures 23 and 24 it may be noted that 12 out of the 15 pairs of sections which can be compared in this way show higher strengths for the Johnson finish than for the standard finish. For instance, the average strength of the 2 slabs represented by the open circles in section 68-J, figure 23, is higher than the average strength of the 3 slabs represented by the open circles in section 67-S, and so on. This would indicate that a part at least of the higher average flexural strength previously noted for the Johnson mix is due to the elimination of excess water and not to honeycomb. This conclusion is substantiated by reference to figure 25, where all of the slabs are free from honeycomb. Four comparisons can be made in this case, all of which show higher strengths for the Johnson finish.

Analyzing figure 23 on the above basis, that is, considering only those slabs which show less than 3 percent honeycomb, we find that 9 slabs in the 4 standard mix gravel sections (nos. 67, 71, 83, and 87) have an average flexural strength of about 720 pounds per square inch as compared to an average of approximately 680 pounds per square inch for 12 slabs in the 4 corresponding Johnson-mix, Johnson-finish sections (nos. 70, 74, 86, and 90). In figure 24 it will be found that 8 slabs in sections 75, 79, 91, and 95 have an average flexural strength of approximately 915 pounds per square inch as against an average of about 890 pounds per square inch for 15 slabs in sections 78, 82, 94, and 98. This is an average difference for the 2 types of aggregate of about 30 pounds per square inch or a little less than 4 percent as compared with a drop, for the control beams representing these sections, of 150 pounds per square inch or roughly 20 percent. A reduction in flexural strength of 150 pounds per square inch corresponds fairly closely to the reduction which would normally be expected from an increase in the water-cement ratio of 0.2, which is approximately the difference between the average water-cement ratio used in the standard mix, standard finish, and the Johnson mix, Johnson finish. (See fig. 20.) These figures show that, entirely aside from the matter of honeycomb and its effect on strength, the Johnson-finish slabs in the leaner mix approached very nearly in strength the standard-finish slabs containing the richer mix.

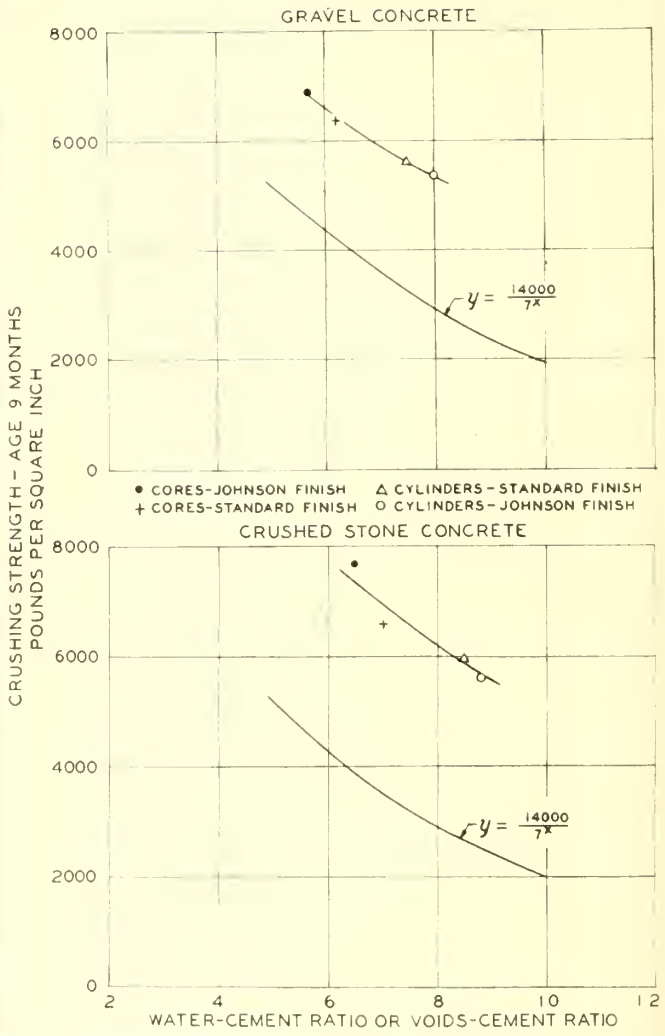


FIGURE 28.—SERIES B, EFFECT OF JOHNSON FINISH; RELATION BETWEEN WATER-CEMENT RATIO, VOIDS-CEMENT RATIO, AND STRENGTH.

SURFACE WEAR TESTS FAVORABLE TO JOHNSON FINISH

The results of wear tests on the slabs in series B are shown graphically in figure 26. The data are plotted so as to show directly the depth of wear of the Johnson-finish slabs in comparison to the standard finish for each proportion and material combination. Each value is the average of 3 tests (1 on each of 2 or 3 separate slabs). It will be observed that, in every case, with the exception of the gravel concrete with coarse sand, the Johnson-finish slabs show appreciably less wear than the standard finish. Just why, in this particular case, the reverse is true is not clear. However, the difference is not very marked, whereas, for several of the other combinations, particularly the crushed stone and coarse sand, and the "Platte River concrete", there is a very distinct advantage in favor of the Johnson sections. The conclusion reached from these tests is that the Johnson method of delayed finishing certainly does not adversely affect the surface hardness of the pavement and very probably tends to increase it, because of the elimination of a portion of the water from the surface.

The results of these tests substantiate the observations which have been made on Johnson-finished pavements in service to the effect that the method of finishing employed in this process thoroughly incorporates the

dry sand-cement mixture into the body of the concrete, and does not cause scaling.

REMOVAL OF SURPLUS WATER HELD MAIN REASON FOR GOOD SHOWING OF JOHNSON METHOD

Values for density and absorption of each material combination and for each method of finishing are shown in figure 27. In the case of series B two values for density are shown, (1) the density of the plastic concrete calculated from the initial water content, and (2) the density of the hardened concrete calculated from the specific gravity of the cores. Inasmuch as the assumption that the plastic concrete is essentially free from air voids does not hold for the "Platte River concrete", values for density calculated from water content have been omitted in this case. Referring now to figure 27 and comparing the density of each standard-finished concrete with the corresponding concrete finished by the Johnson method we find that, whereas the density of the plastic concrete is greater in the case of the mixes which were finished by the standard method, because of the fact that less water was used, the final density is greater in the case of the Johnson-finished concrete. This indicates that the Johnson method removed a considerably larger proportion of the total water before final consolidation took place than was accomplished by the standard method. This ties in very well with the increased crushing strength shown by the Johnson-finished cores.

This is shown graphically in figure 28. In this figure the crushing strength of cylinders representing concrete used in the standard and Johnson-finished sections have been plotted against the corresponding initial water-cement ratios together with the results of core tests on the same concrete plotted against the void-cement ratios which were obtained from the values for density of the hardened concrete. Each point represents the average of 16 tests on 8 sections. It will be observed that, for both coarse aggregate types, all points, both cores and cylinders, lie substantially on a curve paralleling the conventional water-cement ratio strength curve. It should be borne in mind that the core tests were made on the same concrete, insofar as proportions were concerned, as the corresponding cylinder tests. The core tests, however, were made on the concrete after final consolidation had taken place and a considerable amount of water removed. The cylinders, of course, were not subjected to the finishing operation and retained most of the water. These data appear to indicate quite definitely the reason for the increased strength of the cores as compared to the cylinders, as well as the increased strength of the Johnson-finished cores as compared with the standard finish. It is all a matter of the final or net water-cement ratio which, in this case, seems to be measured quite accurately by the actual void-cement ratio of the cores as determined by test.

As indicated above, the "Platte River concrete" cannot be analyzed in this way due to the presence of air voids. Here the actual void-cement ratio of the plastic concrete is much higher than the water-cement ratio. Consequently the density bears no relation to relative water content.

On the assumption that the difference between the initial water-cement ratio and the final void-cement ratio is a measure of water loss during consolidation it will be of interest to compare the average difference for each finishing method and for each type of coarse aggregate. Reference to figure 28 will show that this average difference in the case of the Johnson-finished gravel concrete was 0.23 (from 0.57 to 0.80) as against 0.13 for the standard-finished gravel concrete (0.62 to 0.75). The difference between 0.13 and 0.23, or 0.10, may therefore be said to represent the additional decrease in the volume of voids per unit bulk volume of cement (the void-cement ratio) obtained by the application of the Johnson method. The corresponding value for the crushed-stone concrete is 0.08. On this basis, it seems safe to assume that the use of the Johnson method of finishing is equivalent, insofar as improvement in strength and density are concerned, to a reduction in water-cement ratio of about 0.10.

The effect of this additional increase in density on the ultimate durability of the concrete has not been investigated, although it is planned to make a series of tests along this line as a continuation of the present investigation. All available information on the subject indicates that, for a given mix, a very definite relationship exists between density and durability. It would seem, therefore, that any process which would result in an increase in density would be desirable from this standpoint alone, entirely aside from the effect on strength.

CONCLUDING STATEMENT

The major facts developed by this investigation are summarized in the following paragraphs.

SERIES A: EFFECT OF VIBRATION

1. When finished by the standard method, the minimum slump required to insure pavement slabs substantially free from honeycomb was found to be approximately 2½ inches.

2. When finished by vibration, the minimum slump required to insure the same degree of uniformity was found to be approximately 1 inch. (In the previous investigation it was found that standard-finished pavement slabs constructed of concrete having a 1-inch slump were less uniform in quality than similar slabs in which 2-inch to 3-inch slump concrete was used.)

3. The average flexural strength of all vibrated pavement slabs in which the concrete showed an average slump of 1 inch was somewhat higher than the average strength of the standard-finished concrete of the same proportions in which the average slump was 2 inches. (In the previous investigation standard-finished slabs constructed of 1-inch slump concrete were generally lower in flexural strength than similar concrete having a 2-inch slump.)

4. The average increase noted under (3) was due to the marked increase in strength for the group of sections in which crushed stone was used as coarse aggregate. For the sections containing gravel as coarse aggregate, the vibrated concrete having a 1-inch slump showed a lower average flexural strength than the standard-finished concrete having a 2-inch slump. In the case of slag, the strengths for the two slumps were about the same.

5. The average flexural strength of all vibrated pavement slabs in which the concrete showed an aver-

age slump of ½ inch was lower than the average strength of the standard-finished concrete of the same proportions in which the slump averaged 2 inches.

6. The average decrease noted under (5) was due to the marked decrease in strength for the group of sections in which gravel was used as coarse aggregate. In the case of stone and slag the average strength of the vibrated concrete having a ½-inch slump was about the same as the average strength of the standard-finish sections having a 2-inch slump.

7. The average flexural strength of all vibrated pavement slabs constructed of concrete having the same proportions of cement, fine aggregate, and water but containing one fourth part more coarse aggregate by volume than the standard-finished concrete, was considerably higher than that of the standard-finished concrete. (In the previous investigation, concrete containing more coarse aggregate by volume than the base mix when finished by the standard method showed lower flexural strengths than the base mix, the decrease being roughly proportioned to the amount of coarse aggregate added.)

8. The increase noted under (7) was most marked in the case of the crushed stone concrete and least in the case of the gravel concrete.

9. The average flexural strength of all vibrated concrete slabs containing one half part more coarse aggregate than the standard-finished concrete was somewhat higher than that of the standard-finished concrete. The sections containing crushed stone gave considerably higher values, the sections containing slag slightly higher, and the sections containing gravel lower.

10. The average flexural strength of all vibrated concrete slabs containing three fourths part more coarse aggregate than the standard-finished concrete was lower than that of the standard-finished concrete. The sections containing crushed stone and slag gave about the same results, whereas the sections containing gravel showed a decrease.

11. For concrete having the same water-cement ratio, finishing by vibration did not increase the crushing strength of cores drilled from the pavement slabs containing gravel and slag as coarse aggregate. In the case of crushed stone a considerable increase in crushing strength was noted for the vibrated concrete containing one half and three fourths parts additional coarse aggregate.

12. The process of finishing by vibration did not adversely affect the hardness of the surface.

SERIES B: EFFECT OF DELAYED FINISHING BY THE JOHNSON METHOD

1. Pavement slabs finished by the Johnson method showed higher crushing and flexural strengths than standard-finished concrete of the same proportion but mixed with less water. This observation applies to both of the proportions studied.

2. Pavement slabs finished by the Johnson method developed substantially the same crushing and flexural strengths as standard-finished slabs containing one half part less fine aggregate and one part less coarse aggregate. The Johnson-finished slabs, after correction for the amount of cement in the "dry mix", contained approximately one sack of cement less per cubic yard of concrete than the standard finished slabs.

3. Although, for each proportion, the water-cement ratio at the time of mixing was lower in the case of the standard-finished concrete, the final density of the concrete as revealed by tests on the cores was higher in the case of the Johnson-finished slabs.

4. Variations in crushing and flexural strength due to changes in proportions and methods of finishing were more marked in the crushed stone concrete than in the gravel concrete.

5. The process of finishing by the Johnson method did not adversely affect the hardness of the concrete.

6. The process of finishing by the Johnson method resulted in decreasing the void-cement ratio approximately 0.1 more than the corresponding decrease shown for the standard-finish concrete.

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS I—NATIONAL RECOVERY HIGHWAY PROJECTS
(ON THE FEDERAL-AID HIGHWAY SYSTEM OUTSIDE OF MUNICIPALITIES)

AS OF SEPTEMBER 30, 1933

STATE	NATIONAL RECOVERY FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	National recovery funds	Regular Federal aid	Mileage	Estimated total cost	National recovery funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage	
	$					$	$	$			$		$
Alabama	4.185.067					341,134.93	339.780.69		3.5	26.6	614.613.59	42.1	3.570.453.41
Arizona	3.804,731					157,051.86	78.525.93	78,525.93	1.3	.2	915,672.59	59.8	2,549.277.72
Arkansas	3,374.167										189.564.16	3.3	3.106.076.91
California	7.803,677					91,922.04	55,464.33			.4	4,780.084.60	184.7	2.968.128.07
Colorado	3,437,265					487,554.21	487,554.21		4.3	31.0	634,513.71	24.2	2.315.197.08
Connecticut	1.404.213										202.625.72	2.9	1.201,587.28
Delaware	909,544					113,840.45	113.840.45		1.8	1.2	352.476.85	20.7	443.226.70
Florida	2.615,917										881,371.61	56.2	1.734.545.39
Georgia	5.045.592												5.045.592.00
Idaho	2.243,125					1,047,250.42	1,047.151.42		6.2	86.5	417,428.20	25.4	778.545.38
Illinois	4,431.348										85.811.02	.9	4.345.536.98
Indiana	4,717.786										2.272.727.86	80.3	2,445.058.14
Iowa	5,027.830					2,044,459.11	1,923.800.00		11.1	93.1	1,792.400.00	94.7	1.311.630.00
Kansas	5,044,802										2.216,413.22	266.7	2.828.388.78
Kentucky	3,608.332										1.568.028.46	134.2	2.040.303.54
Louisiana	2,914.295					419,189.70	419,189.70			18.0	926.685.80	9.1	1,568.419.50
Maine	1,684.959					538,434.87	538.434.87		13.7	22.3	276.778.22	5.0	869.745.91
Maryland	1,782,263										173,428.09	4.8	1.608.834.91
Massachusetts	1.932,950					266,711.51	137.412.16	124.984.24	32.0	7.4	58.525.35	5.7	1,737,012.49
Michigan	5.094.491					660,500.00	660.500.00		4.7	27.7	700.000.00	57.3	3.733.991.00
Minnesota	5.115,153					1,072,674.82	1,072,674.82		14.1	171.6	995.796.77	182.0	3,046.681.41
Mississippi	3.489.337					195,020.80	105.718.33	89.302.47		8.9	106,530.65	9.6	3,277,088.02
Missouri	6,090,153					447,056.18	388,331.43		1.0	13.8	2.567.159.51	101.9	3.134.662.06
Montana	3.719.874					1,784,361.58	1.604.361.58	100,000.00	8.9	130.1	1,029.066.77	90.0	1,086.445.65
Nebraska	3.914,481					1,761,592.34	1,402.902.80		13.0	152.9	1.139.126.46	137.4	1,372.451.74
Nevada	2,909,387					142,181.52	142,181.52		8.4	26.0	371,808.43	65.8	2.395.397.05
New Hampshire	954.919					143,242.86	110.000.00		.9	2.4	212,892.65	5.2	632.026.35
New Jersey	3.065.137										374.227.67	6.1	2.690,909.33
New Mexico	2.896,467					722,971.10	722.971.10		6.8	100.5	971,567.19	107.7	1.201,928.71
New York	10.830.099					4,456,800.00	4.216.610.00	240.000.00	4.1	96.1	4.415.040.00	83.3	2.198.449.00
North Carolina	4,761,147					598,096.51	299.048.29	299.048.22	2.0	70.5	226,944.44	66.0	4,235.154.27
North Dakota	2.902.224					338,303.54	338,303.54		2.1	142.3	1.392,522.76	461.8	1.171.397.70
Ohio	6.968.066					1,420,820.00	1,420,820.00		1.0	42.1	3.893.845.00	84.4	1,653,401.00
Oklahoma	4.608.399					22,165.59	22.165.59		4.5	3.8	2.971,916.39	175.9	1,614.317.02
Oregon	3.053.448					1,097,865.86	1,060.851.85		10.3	82.2	1,144.892.10	46.6	847.704.05
Pennsylvania	5.757.978										1.627.200.00	44.2	4.130.778.00
Rhode Island	999.354					233,128.00	233,128.00		1.3	6.6	733.837.82	13.3	32.385.18
South Carolina	2,729,583					74,882.33	72.152.71	2.729.62		19.2	1.135.439.20	125.1	1.521.991.09
South Dakota	3.005.739					546,992.49	546.992.49		7.9	73.6	904.465.42	147.2	1,554.281.09
Tennessee	4,246,309										429,668.21	15.5	3.816.640.79
Texas	12,122.012					1,013,160.70	801,491.63		3.1	81.4	2,878.310.37	384.1	8.442.210.00
Utah	2,097.354					1,104,247.51	1,104,247.51		34.3	136.9	404.077.41	28.9	589.029.08
Vermont	931.919					553,013.21	553.013.21		6.3	25.3	1.949.75	.4	376.956.04
Virginia	3,708.379					69,463.18	69,463.18			2.3	824,678.48	40.3	2.814,237.34
Washington	3.057.934					841,058.58	837.701.76	3.356.82	6.0	35.5	866,984.93	32.3	1,353.247.31
West Virginia	2,013,405					573,068.56	573,068.56		1.0	24.4	1,138.827.63	42.9	301,508.81
Wisconsin	4,862,441					1,781,909.98	1.760,409.98	21,500.00	22.2	71.0	843,838.61	36.4	2.258.192.41
Wyoming	2,250,663					911,611.13	882.769.08		11.0	184.1	1,098,878.29	221.5	269.015.63
District of Columbia													
Hawaii	1.683,956										43.500.88	5.1	1,640.455.12
TOTALS	185.807.671					28,074,537.47	26,143,032.72	959,447.30	9.0	2.017.9	53.804.142.84	3.838.9	105.860.495.44

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS II—NATIONAL RECOVERY MUNICIPAL HIGHWAY PROJECTS
(ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES)

AS OF SEPTEMBER 30, 1933

STATE	NATIONAL RECOVERY FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	National recovery funds	Regular Federal aid	Mileage	Estimated total cost	National recovery funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage	
	$					$	$				$		$
Alabama	2,092,533										21,544.05	1.7	2,070,988.95
Arizona	781,794					5,345.26	5,345.26			0.4	6,631.04	.6	769,817.70
Arkansas	1,687,084										32,936.12	.5	1,654,147.88
California	3,901,839					38,291.40	33,871.86			.8	291,255.96	3.3	3,576,711.18
Colorado	1,718,633					100,849.92	100,849.92		2.0	2.7	4,106.96	.7	1,613,676.12
Connecticut	802,407										380,970.16	3.2	421,436.84
Delaware	454,772					30,801.70	30,801.70			.7	39,357.50	.2	384,612.80
Florida	1,307,959										76,807.52	2.9	1,231,151.48
Georgia	2,724,620												2,724,620.00
Idaho	1,121,562					38,360.76	38,360.76		5.2	2.9	1,112.10	.1	1,082,089.14
Illinois	6,877,199												6,877,199.00
Indiana	4,818,165												4,818,165.00
Iowa	2,815,585					139,570.61	131,400.00		11.4	3.9	642,400.00	18.2	2,041,785.00
Kansas	2,522,401					21,868.02	21,868.02				57,694.37	6.4	2,442,838.11
Kentucky	2,029,687										31,123.23	1.5	1,998,563.77
Louisiana	1,457,148										379,759.02	9.3	1,077,388.98
Maine	842,479					70,960.63	70,960.63		28.2	2.6	141,049.29	2.0	630,469.08
Maryland	891,132										13,706.65	.6	877,425.35
Massachusetts	4,136,382					271,359.00	161,737.14	103,375.00	4.3	2.6	726,380.47	4.4	3,248,264.39
Michigan	4,457,679					24,000.00	24,000.00			.9	229,700.00	7.4	4,203,979.00
Minnesota	3,410,102					439,204.01	439,204.01		3.9	22.3	831,452.70	29.3	2,139,445.29
Mississippi	1,744,669										17,357.43	1.8	1,727,311.57
Missouri	3,045,077					29,333.57	29,333.57			.7	615,389.36	7.0	2,400,354.07
Montana	1,859,937					5,275.14	5,275.14		75.8	.4	25,871.92	2.3	1,828,789.94
Nebraska	1,957,240					103,831.27	103,831.27		5.8	5.0	113,436.30	7.2	1,739,972.43
Nevada	500,051					57,689.61	57,689.61		5.2	1.2			442,361.39
New Hampshire	477,460										20,000.00	.3	457,460.00
New Jersey	3,217,442										1,215,595.66	7.8	2,001,846.34
New Mexico	1,448,234										72,429.57	2.3	1,375,804.43
New York	7,837,865					1,214,070.00	1,214,070.00		.7	8.0	2,553,200.00	23.1	4,070,595.00
North Carolina	2,380,573					22,158.73	22,158.73			1.1	79,888.38	7.9	2,278,525.89
North Dakota	1,451,112					7,557.18	7,557.18			3.4	15,409.26	3.1	1,428,145.56
Ohio	4,645,378					54,350.00	54,350.00			1.0	766,365.00	10.5	3,824,663.00
Oklahoma	2,304,200					3,272.08	3,272.08			.5	51,564.75	4.1	2,249,363.17
Oregon	1,526,724										218,414.35	4.6	1,308,309.65
Pennsylvania	5,416,051										364,900.00	9.0	5,051,151.00
Rhode Island	499,677												499,677.00
South Carolina	1,364,791					6,542.28	6,137.27	405.01		1.6	79,228.79	7.5	1,279,424.94
South Dakota	1,502,870					21,164.62	21,164.62			2.1	30,130.16	3.3	1,451,575.22
Tennessee	2,123,155										26,417.32	1.3	2,096,737.68
Texas	6,061,006					28,593.48	28,593.48			.4	185,110.13	22.7	5,847,302.39
Utah	1,048,677					191,813.07	180,653.07		8.9	5.4	297,055.21	8.0	570,968.72
Vermont	470,628					13,707.96	13,707.96			.9			456,920.04
Virginia	1,854,189										90,385.97	2.6	1,763,803.03
Washington	1,877,571					37,211.64	37,211.64		26.9	1.5	103,981.60	2.6	1,736,377.76
West Virginia	1,342,270					39,929.76	39,929.76			.4	96,749.27	.5	1,205,590.97
Wisconsin	2,431,220					368,256.89	368,256.89		8.1	12.5	327,398.62	6.3	1,735,564.49
Wyoming	1,125,332										267,476.57	5.1	857,855.43
District of Columbia	1,151,081					169,320.65	169,320.65		4.1	1.1	748,503.00	2.8	233,257.35
Hawaii													
TOTALS	113,515,642					3,554,689.24	3,420,912.22	103,780.01	4.3	87.0	12,290,246.26	246.0	97,804,483.52

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION

AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS III—NATIONAL RECOVERY SECONDARY HIGHWAY PROJECTS

(On Secondary or Feeder Roads not now included in the approved system of Federal-aid highways, but which are either part of the State highway system or are important local highways leading to shipping points, or which will permit the coordination or extension of existing transportation facilities)

AS OF SEPTEMBER 30,1933

STATE	NATIONAL RECOVERY FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	National recovery funds	Mileage	Estimated total cost	National recovery funds allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage		
	$				$	$			$		$	
Alabama	2,092,533										2,092,533.00	Alabama
Arizona	625,435										625,435.00	Arizona
Arkansas	1,687,084										1,687,084.00	Arkansas
California	3,901,838								235,928.19	5.5	3,665,909.81	California
Colorado	1,718,632										1,718,632.00	Colorado
Connecticut	659,120										659,120.00	Connecticut
Delaware	454,772										454,772.00	Delaware
Florida	1,307,958										1,307,958.00	Florida
Georgia	2,320,973										2,320,973.00	Georgia
Idaho	1,121,562				11,625.34	10,713.08		2.8			1,110,848.92	Idaho
Illinois	6,262,223										6,262,223.00	Illinois
Indiana	501,892										501,892.00	Indiana
Iowa	2,212,245				3,718.11	3,500.00		.6	514,700.00	33.8	1,694,045.00	Iowa
Kansas	2,522,401								12,972.27	3.8	2,509,428.73	Kansas
Kentucky	1,879,340								364,518.00	33.3	1,514,822.00	Kentucky
Louisiana	1,457,148								59,390.34	2.3	1,397,757.66	Louisiana
Maine	842,479				703,465.36	703,465.36	15.1	83.3	137,934.64	10.6	1,079.00	Maine
Maryland	891,132										891,132.00	Maryland
Massachusetts	527,768				384,331.26	384,331.26	6.2	11.5			143,436.74	Massachusetts
Michigan	3,184,057										3,184,057.00	Michigan
Minnesota	2,131,314				114,079.73	114,079.73	.9	14.6	413,755.16	27.3	1,603,479.11	Minnesota
Mississippi	1,744,669										1,744,669.00	Mississippi
Missouri	3,045,076								424,647.24	86.4	2,620,428.76	Missouri
Montana	1,859,937										1,859,937.00	Montana
Nebraska	1,957,240								308,248.08	19.5	1,648,991.92	Nebraska
Nevada	1,136,479								196,097.57	14.4	940,381.43	Nevada
New Hampshire	477,460										477,460.00	New Hampshire
New Jersey	63,460										63,460.00	New Jersey
New Mexico	1,448,234										1,448,234.00	New Mexico
New York	3,662,137				1,509,141.00	1,505,620.00	2.7	34.7	1,578,390.00	27.8	578,127.00	New York
North Carolina	2,380,573				68,099.79	68,099.79		9.3	86,666.14	7.6	2,225,807.07	North Carolina
North Dakota	1,451,112										1,451,112.00	North Dakota
Ohio	3,871,148								151,064.90	1.2	3,720,083.10	Ohio
Oklahoma	2,304,199										2,304,199.00	Oklahoma
Oregon	1,526,724								217,862.70	52.6	1,308,861.30	Oregon
Pennsylvania	7,716,975										7,716,975.00	Pennsylvania
Rhode Island	499,677										499,677.00	Rhode Island
South Carolina	1,364,791								322,186.38	41.6	1,042,604.62	South Carolina
South Dakota	1,502,870								12,525.42	3.8	1,490,344.58	South Dakota
Tennessee	2,123,155										2,123,155.00	Tennessee
Texas	6,061,006				83,781.20	83,781.20		12.9	566,063.55	124.5	5,411,161.25	Texas
Utah	1,048,677				19,060.36	19,060.36	31.5	7.0	39,640.75	1.0	989,975.89	Utah
Vermont	465,026										465,026.00	Vermont
Virginia	1,854,189								158,524.75	11.6	1,695,664.25	Virginia
Washington	1,180,362				223,949.30	223,949.30		9.2	77,925.76	14.4	878,486.94	Washington
West Virginia	1,118,559										1,118,559.00	West Virginia
Wisconsin	2,431,220				148,927.19	148,927.19	.7	13.1	542,518.16	37.7	1,739,774.65	Wisconsin
Wyoming	1,125,332										1,125,332.00	Wyoming
District of Columbia	767,388				62,897.00	62,897.00	1.6	1.5	262,447.00	1.9	442,044.00	District of Columbia
Hawaii	187,106										187,106.00	Hawaii
TOTALS	94,676,687				3,333,075.64	3,328,424.27	5.4	200.5	6,684,007.00	562.6	84,664,255.73	TOTALS

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

SUMMARY OF CLASSES I, II, AND III

AS OF SEPTEMBER 30,1933

STATE	TOTAL APPORTIONMENT OF NATIONAL RECOVERY FUNDS	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR ALL NEW PROJECTS
		Total cost	National recovery funds	Regular Federal aid	Mileage	Estimated total cost	National recovery funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage	
	$					$	$	$			$		$
Alabama	8,370,133										636,157.64	43.8	7,733,975.36
Arizona	5,211,960					347,280.19	345,125.95		3.5	27.0	922,303.63	60.4	3,944,530.42
Arkansas	6,748,335					157,051.86	78,525.93	78,525.93	1.3	.2	222,500.28	3.8	6,447,308.79
California	15,607,354					130,213.44	89,336.19		1.2		5,307,268.75	193.5	10,210,749.06
Colorado	6,874,530					588,404.13	588,404.13		3.9	33.7	638,620.67	24.9	5,647,505.20
Connecticut	2,365,740										583,595.88	6.1	2,282,144.12
Delaware	1,319,088					144,642.15	144,642.15		1.4	1.9	391,834.35	20.9	1,282,611.50
Florida	5,231,834										958,179.13	59.1	4,273,654.87
Georgia	10,091,185												10,091,185.00
Idaho	4,486,249					1,097,236.52	1,096,225.26		6.1	92.3	418,540.30	25.5	2,971,483.44
Illinois	17,570,770										85,811.02	.9	17,484,958.98
Indiana	10,037,843										2,272,727.86	80.3	7,765,115.14
Iowa	10,055,660					2,187,747.83	2,058,700.00		1.1	97.6	5,949,500.00	146.7	5,047,460.00
Kansas	10,089,604					21,868.02	21,868.02				2,287,080.36	276.8	7,780,655.62
Kentucky	7,517,359										1,963,669.69	169.0	5,553,689.31
Louisiana	5,828,591					419,189.70	419,189.70			18.0	1,365,835.16	20.7	4,043,566.14
Maine	3,369,917					1,312,860.86	1,312,860.86		15.2	108.3	555,762.15	17.6	1,501,293.99
Maryland	3,564,527										187,134.74	5.4	3,377,392.26
Massachusetts	6,597,100					922,401.77	683,480.56	228,359.24	11.0	21.5	784,905.82	10.2	5,128,713.62
Michigan	12,736,227					684,500.00	684,500.00		4.5	28.6	929,700.00	64.7	11,122,027.00
Minnesota	10,656,569					1,625,958.56	1,625,958.56		10.4	208.5	2,241,004.63	238.6	6,789,605.81
Mississippi	6,978,675					195,020.80	105,718.33	89,302.47		8.9	123,888.08	11.3	6,749,068.59
Missouri	12,180,306					476,389.75	417,665.00		1.0	14.5	3,607,196.11	195.4	8,155,444.89
Montana	7,439,748					1,789,636.72	1,609,636.72	100,000.00	9.1	130.5	1,054,938.69	92.4	4,775,172.59
Nebraska	7,828,961					1,865,423.61	1,506,734.07		12.5	157.9	1,560,810.84	164.2	4,761,416.09
Nevada	4,545,917					199,871.13	199,871.13		7.5	27.1	567,906.00	80.2	3,778,139.87
New Hampshire	1,909,839					143,242.86	110,000.00		.9	2.4	232,892.65	5.5	1,566,946.35
New Jersey	6,346,039										1,589,823.33	13.9	4,756,215.67
New Mexico	5,792,935					722,971.10	722,971.10		6.8	100.5	1,043,996.76	109.9	4,025,967.14
New York	22,330,101					7,180,011.00	6,936,300.00	240,000.00	3.2	138.8	8,546,630.00	134.2	6,847,171.00
North Carolina	9,522,293					688,355.03	389,306.81	299,048.22	1.5	80.9	393,498.96	81.5	8,739,487.23
North Dakota	5,804,448					345,860.72	345,860.72		2.0	145.7	1,407,932.02	464.9	4,050,655.26
Ohio	15,484,592					1,475,170.00	1,475,170.00		.9	43.1	4,811,274.90	96.1	9,198,147.10
Oklahoma	9,216,798					25,437.67	25,437.67		3.9	4.2	3,023,481.14	180.0	6,167,879.19
Oregon	6,106,896					1,097,865.86	1,060,851.85		10.3	82.2	1,581,169.15	103.7	3,464,875.00
Pennsylvania	18,891,004										1,992,100.00	53.2	16,898,904.00
Rhode Island	1,998,708					233,128.00	233,128.00		1.3	6.5	733,837.82	13.3	1,031,742.18
South Carolina	5,459,165					81,424.61	78,289.98	3,134.63		20.9	1,536,854.37	174.2	3,844,020.65
South Dakota	6,011,479					568,157.11	568,157.11		7.6	75.7	947,121.00	154.3	4,496,200.89
Tennessee	8,492,619										456,085.53	16.8	8,036,533.47
Texas	24,244,024					1,125,535.38	913,866.31		2.7	94.8	3,629,484.05	531.3	19,700,673.64
Utah	4,194,708					1,315,120.94	1,303,960.94		30.8	149.3	740,773.37	37.8	2,149,973.69
Vermont	1,867,573					566,721.17	566,721.17		6.2	26.2	1,949.75	.4	1,298,902.08
Virginia	7,416,757					69,463.18	69,463.18			2.3	1,073,589.20	54.5	6,273,704.62
Washington	6,115,867					1,102,219.52	1,098,862.70	3,356.82	5.5	46.2	1,048,892.29	49.3	3,968,112.01
West Virginia	4,474,234					612,998.32	612,998.32		1.0	24.8	1,235,576.90	43.4	2,625,658.78
Wisconsin	9,724,881					2,299,094.06	2,277,594.06	21,500.00	18.5	96.6	1,713,755.39	80.5	5,733,531.55
Wyoming	4,501,327					911,611.13	882,769.08		11.0	184.0	1,366,354.86	226.6	2,252,203.06
District of Columbia	1,918,469					232,217.65	232,217.65		3.4	2.6	1,010,950.00	4.7	675,301.35
Hawaii	1,871,062										43,500.88	5.1	1,827,561.12
TOTALS	394,000,000					34,962,302.35	32,892,369.21	1,063,227.31	8.1	2,305.4	72,778,396.10	4,647.5	288,329,234.69

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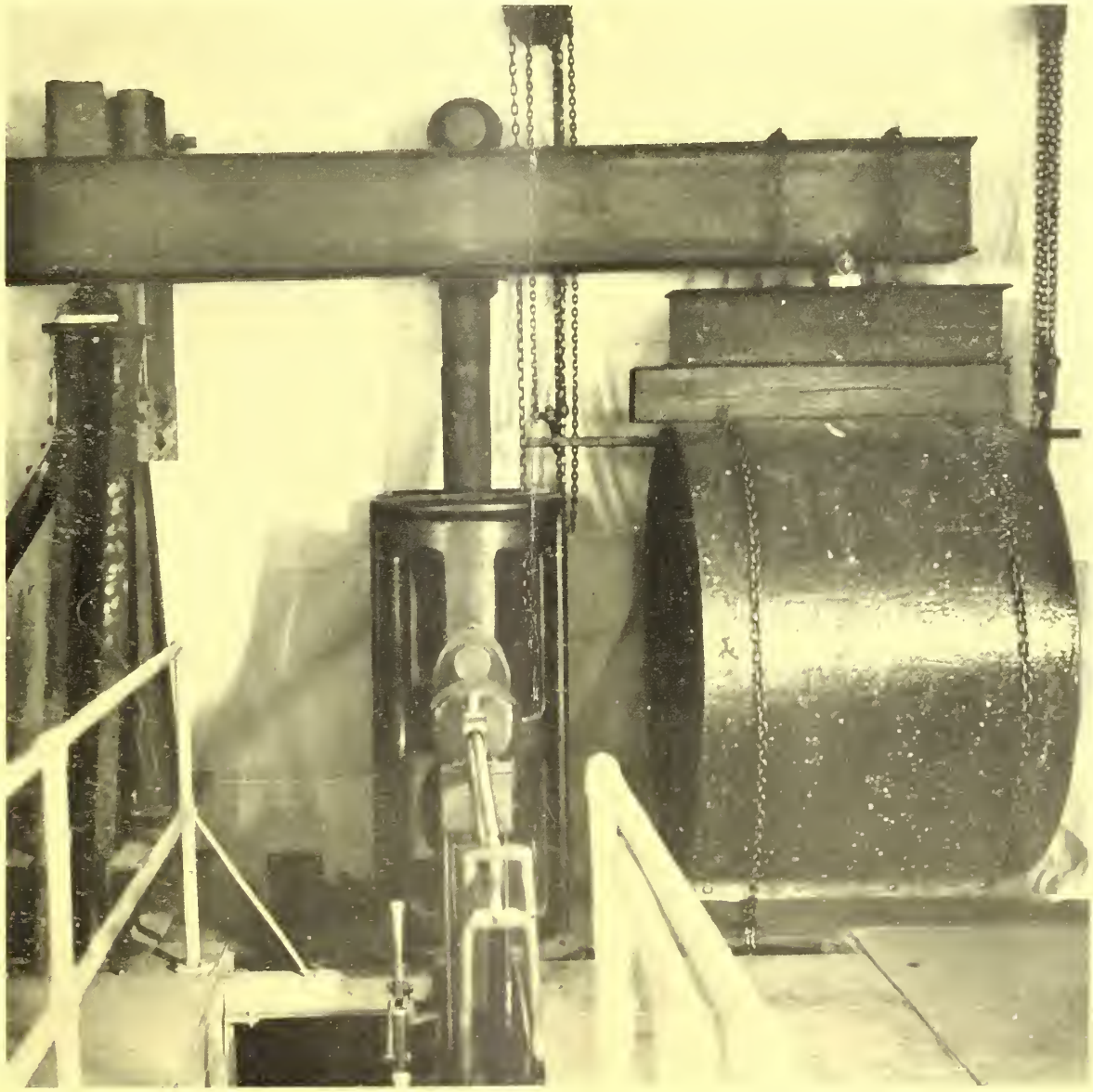
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 9 NOVEMBER 1933



APPARATUS FOR TESTING CAST-IRON CULVERT PIPE

For sale by the Superintendent of Documents, Washington, D.C. - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 9 November 1933

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

STRENGTH TESTS OF CAST-IRON CULVERT PIPE

Reported by E. F. KELLEY, Chief, Division of Tests, and W. F. KELLERMANN, Associate Materials Engineer, United States Bureau of Public Roads

THE SPECIFICATIONS for cast-iron culvert pipe which are in common use in highway work are greatly lacking in uniformity, and show a wide range in the requirements for strength and thickness of metal for pipe which presumably are to be subjected to similar conditions of service.

INCONSISTENCIES FOUND IN SPECIFICATIONS

Of 36 State highway department specifications current in 1931, 2 required a minimum strength in the 3-edge bearing test but did not specify thickness of metal; 7 specified minimum thickness of metal but gave no strength requirements; and 27 specified both minimum strength and minimum shell thickness. In the specifications of the latter group the requirements for strength and shell thickness were not always consistent.

The minimum requirements for strength in the 3-edge bearing test ranged from 1,500 pounds per foot of diameter per linear foot of pipe (1,500 D) to 3,000 pounds per foot of diameter per linear foot of pipe (3,000 D). Fourteen States favored 1,500 D for all sizes of pipe, 4 States specified 1,800 D, 7 States required 2,000 D and 1 State required 3,000 D. Three States specified 1,500 D for pipe over 24 inches in diameter, and 2,000 D for pipe under 24 inches in diameter.

The diversity of the requirements for minimum shell thickness of smooth or plain cast-iron pipe is shown graphically in figure 1. Thus the minimum thickness for 12-inch pipe ranged from 0.31 to 0.54 inch and for 48-inch pipe the range was from 0.75 to 1.38 inch, a difference of nearly 100 percent. The influence of the standard requirements for cast-iron water pipe are also shown in figure 1, 2 States having specified for culvert pipe the thicknesses required for class A water pipe in the sizes up to and including 36-inch and 1 State specified the thickness required for the larger sizes of 42 inches and 48 inches.

The solid lines in figure 1 show the theoretical relationship between shell thickness and strength in the 3-edge bearing test, for smooth cast-iron pipe without hubs, for an assumed modulus of rupture of 35,000 pounds per square inch. This modulus of rupture is an approximate average figure for modulus of rupture obtained in tests of sand-cast pipe. The dotted lines show the theoretical thicknesses increased by 10 percent, an allowance suggested to provide for unavoidable inaccuracy in manufacture. It is interesting to observe that the minimum thicknesses commonly specified for the smaller sizes of pipes were generally in excess of those which would be required for a load of 3,000 D pounds per linear foot, while in the larger sizes the required thicknesses, in many cases, were below those which would be necessary to insure a load capacity of 2,000 D. It is believed that a strength of 2,000 D in the 3-edge bearing test is the minimum which should be required for cast-iron pipe for use in culverts, yet 21 States, in 1931, specified a minimum shell thickness of 0.69 inch for 36-inch pipe, which is too low to insure this strength in sand-cast pipe.

An example of the lack of relationship between requirements for strength and shell thickness was found in the specification of one State which required a minimum load of 1,500 D for pipe having the thickness of class A water pipe. Pipe of this weight is generally more than adequate for a load of 3,000 D. Another State specification required a minimum strength of 3,000 D but, for the larger sizes of pipe, specified minimum shell thicknesses insufficient for a load of 2,000 D.

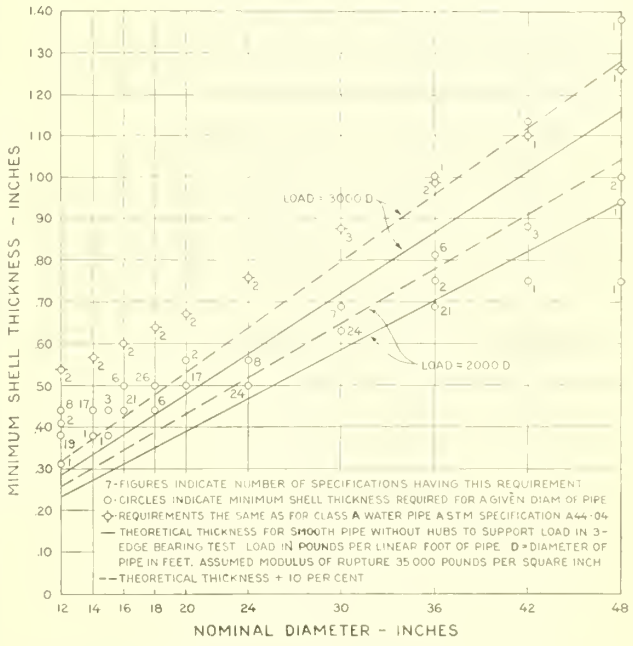
MINIMUM SHELL THICKNESS - INCHES
NOMINAL DIAMETER - INCHES
LOAD = 3000 D
LOAD = 2000 D
7-FIGURES INDICATE NUMBER OF SPECIFICATIONS HAVING THIS REQUIREMENT
O-CIRCLES INDICATE MINIMUM SHELL THICKNESS REQUIRED FOR A GIVEN DIAM OF PIPE
REQUIREMENTS THE SAME AS FOR CLASS A WATER PIPE A.S.T.M. SPECIFICATION A44-04
— THEORETICAL THICKNESS FOR SMOOTH PIPE WITHOUT HUBS TO SUPPORT LOAD IN 3-EDGE BEARING TEST. LOAD IN POUNDS PER LINEAR FOOT OF PIPE. D=DIAMETER OF PIPE IN FEET. ASSUMED MODULUS OF RUPTURE 35,000 POUNDS PER SQUARE INCH
--THEORETICAL THICKNESS + 10 PER CENT

FIGURE 1.—RELATION BETWEEN MINIMUM SHELL THICKNESS AND DIAMETER OF SMOOTH CAST-IRON CULVERT PIPE AS SPECIFIED BY 34 STATES IN 1931.

The testing of cast-iron culvert pipe to determine compliance with specification requirements for strength is an expensive procedure, particularly if the ultimate strength of the test specimens is determined, thereby destroying the pipe. With a sufficient knowledge of the strength characteristics of commercial cast-iron pipe, as influenced by shell thickness and diamater, it should be possible to eliminate much of this expense and to accept pipe on the basis of thickness and weight more generally than has been customary in the past. It was largely on this account that the tests described in this report were undertaken, but the data which have been developed should also prove useful in indicating revisions which might be made in existing specifications in the interest of uniformity and consistency.

The culvert pipe tested were as follows:[1]

Smooth, sand-cast pipe, furnished by one manufacturer in sizes from 12- to 48-inch, and by another manufacturer in sizes from 12- to 36-inch.

[1] Acknowledgment is made to the United States Pipe and Foundry Co., the American Casting Co. and the Alabama Pipe Co. for the specimens of culvert pipe which were used in these tests.

Smooth pipe cast by a centrifugal process in metal contact molds, in the sizes 12-, 16-, and 20-inch.

Spiral-corrugated pipe in sizes from 15- to 36-inch.

Ribbed pipe in the sizes 15-, 18-, and 24-inch.

In general three specimens of each size and type were tested. Six specimens of each size of smooth sand-cast pipe and of smooth centrifugally cast pipe were supplied, 3 specimens had bells or hubs and 3 had plain ends. There were 6 specimens of each size of spiral-corrugated pipe, 3 specimens representing the pipe section and 3 representing the cuff section. All specimens had a net length of approximately 3 feet, exclusive of the overlapping portion of end fittings. Some had been cast in this length and the others were cut from longer pipe. The smooth pipe with plain ends were cut from pipe which had been cast with hubs.

The spiral-corrugated pipe had a relatively thin shell which was cast in the form of spiral circumferential corrugations. Sections of pipe are fastened together by screwing one into the other. Since each section of pipe is of uniform diameter, the inner, or pipe section, is of slightly smaller diameter than the outer or cuff section.

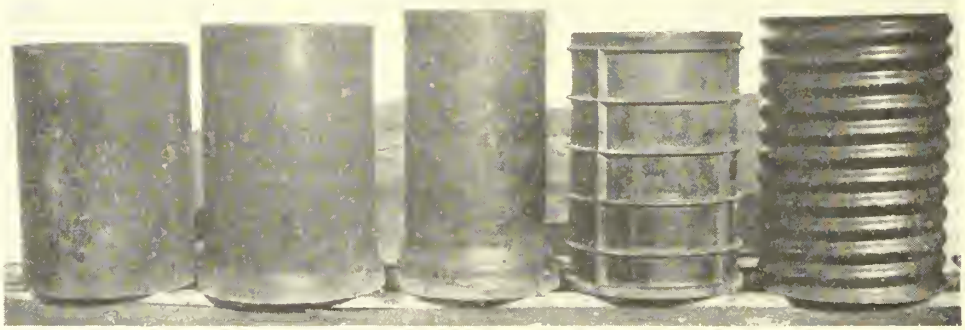

FIGURE 2.—REPRESENTATIVE SPECIMENS OF TYPES OF PIPE TESTED. SPECIMENS ARE (LEFT TO RIGHT) SMOOTH SAND-CAST PIPE PLAIN END; SMOOTH SAND-CAST PIPE, BELL END; SMOOTH CENTRIFUGALLY CAST PIPE, BELL END; RIBBED PIPE; SPIRAL CORRUGATED PIPE.

The ribbed pipe had a relatively thin shell reinforced by four longitudinal ribs, and by transverse ribs, the spacing of which varied somewhat with the size of the pipe.

Representative specimens of the different types of pipe are illustrated in figure 2.

SPECIFICATIONS VARY AS TO POSITION OF LOAD

All pipe were tested for strength in the 3-edge bearing test. This test has been used on sewer pipe and culvert pipe for a number of years and has become an accepted standard but it has not been completely standardized in one important respect. This is frequently overlooked and merits some discussion.

The general set-up for the test is shown in figure 3. The lower bearing consists of a rigid block to which are securely fastened two longitudinal wooden strips having vertical sides with the interior top corners rounded to a radius of approximately one half inch. The earlier specifications required that these strips be spaced 1 inch apart while the more recent ones provide that the space between them shall be not less than one half inch or more than 1 inch for each foot of nominal pipe diameter. The upper bearing, through which the load is applied, is a rigid wooden block. In the case of pipe with plain ends the upper and lower bearings extend the full length of the pipe while for pipe with bell ends the bearings extend from the spigot end to the point where the barrel begins to flare out to meet the bell. It is with respect to the position of the point or center of loa application, for pipes with bell ends, that the variou standard and tentative standard specifications diffe and this is important since the position of the load ha a considerable influence on the ultimate strength devel oped in the test.

Three positions of the load are used in various speci fications:

(*a*) Load at the center of the upper bearing block.

(*b*) Load at the center of the laying-length of th pipe. The laying-length of pipe is the distance fron the spigot end to the inside shoulder of the bell an therefore is a length intermediate between the lengt of the upper bearing block and the full length of th pipe, including the bell.

(*c*) Load at the center of the length of the pipe a shown at the top of figure 3. This position results i less eccentricity of load application than the other tw and is required by the most recent specifications This position of load, with a few exceptions which wil be mentioned later, was used in these tests.

Position (*a*) is required by the A.S.T.M. (Americar Society for Testing Materials) Standard Specification for Clay Sewer Pipe, C13-24 and A.S.T.M. Standar Specifications for Cement-Concrete Sewer Pipe, C14-24

Position (*b*) is required by the 1926 Tentative Stand ard Specifications for Reinforced Concrete Culver Pipe of the Joint Concrete Culvert Pipe Committe and by the Tentative Standard Methods of Sampling and Testing Highway Materials of the American Asso ciation of State Highway Officials as published in Unite States Department of Agriculture Bulletin No. 1216 Both of these have since been revised but the latter i still used to some extent.

Position (*c*) is required by the 1928 Tentative Stand ard Specifications of the Joint Concrete Culvert Pip Committee; by the A.S.T.M. Tentative Standar Specifications for Reinforced-Concrete Culvert Pipe C76-30T, which, in substance, are the same as the 192 specifications of the Joint Committee; by the A.S.T.M Tentative Standard Specifications for Reinforced Concrete Pipe, C75-30T; and by the 1931 edition of th Tentative Standard Specifications for Highway Mate rials and Methods of Sampling and Testing of th American Association of State Highway Officials which is a revision of Department of Agriculture Bulle tin No. 1216.

The State specifications for making strength tests discussed above, referred variously to A.S.T.M. Speci fications C13-24 and C14-24 and to Bulletin No. 1216 o the Department of Agriculture. Some specified the 3-edge bearing test without describing it, others gav

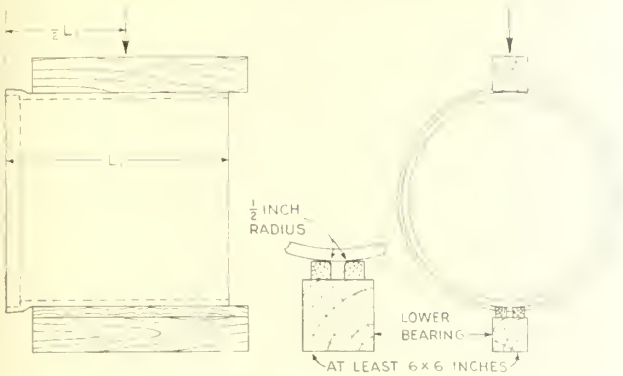
½ INCH RADIUS
LOWER BEARING
AT LEAST 6×6 INCHES
3-EDGE BEARINGS FOR PIPE WITH BELL END
½ INCH RADIUS
LOWER BEARING
AT LEAST 6×6 INCHES
3-EDGE BEARINGS FOR PIPE WITHOUT BELL END

FIGURE 3.—POSITION OF BEARINGS IN CAST-IRON PIPE TESTS.

incomplete descriptions which did not define the position of load, another fairly large group required the test for strength to be made "in the manner prescribed by Committee C-4, American Society for Testing Materials." The latter is a particularly ambiguous requirement since, prior to 1932, all of the A.S.T.M. specifications to which reference has been made were under the jurisdiction of Committee C-4 and that, in addition to the 3-edge bearing test, these specifications also recognize the 2-edge bearing test and the sand-bearing test.

In conducting the 3-edge bearing test on the specimens included in this investigation, the distance between the longitudinal strips of the lower bearing was varied as shown in the following table:

Size of pipe	Distance between strips
Inches	*Inches*
12	1
15	1
16	1
18	1
20	2
24	2
30	2
36	3
42	3
48	3

The test load was applied to the upper bearing block at the center of the length of the pipe with the exception of four specimens of 12-inch pipe with bell ends. Three of these were sand-cast pipe furnished by manufacturer A and one was a specimen of centrifugally cast pipe (tables 1 and 3). In the case of the exceptions

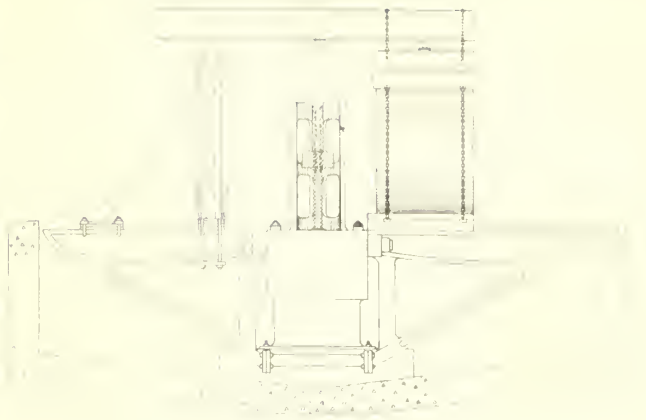

FIGURE 4.—FORTY-EIGHT-INCH PIPE SET UP IN 200,000-POUND TESTING MACHINE.

mentioned, the load was applied at the center of the length of bearing block and therefore was eccentric with respect to the center of length of the pipe.

TESTING APPARATUS DESCRIBED

Since a machine designed particularly for pipe testing was not available, the 3-edge bearing tests were made in a 200,000-pound universal testing machine having two extensions, or wings, attached to opposite sides and forming a part of the weighing table. The general arrangement of the testing apparatus is shown in figure 4. The cover page shows a 48-inch plain-end pipe ready for testing. The pipe was placed on one extension of the weighing table and on the opposite extension there was placed a vertical steel member of suitable height which served as a support for the loading beam and also as a counter weight. Two I-beams extending across the weighing table were supported on one side by a knife-edge bearing block on top of the vertical steel member and on the other side by a roller bearing which rested on the upper bearing block on the pipe. The load was applied through an eyebar and pin, the pin being supported by the I-beams at mid-span between the two bearings and the eyebar extending down through the movable head of the testing machine. A nut and washer on the lower end of the eyebar made contact with the movable head as it was lowered and thus transmitted load to the two bearings of the I-beams. With this arrangement the test load on the pipe was one half the load indicated on the weighing beam of the testing machine. During application of the load, the testing machine was run in low gear which corresponded to an idle crosshead speed of 0.05 inch per minute.

Straight bearing blocks of the same type were used on all types of pipe. This resulted in a much smaller area of contact on the special types of pipe than on the smooth pipe. On the corrugated pipe the applied load was concentrated on the upper surfaces of the corrugations and on the ribbed pipe the load was concentrated on the transverse ribs, the specimens being arranged for the test so that no load was applied to the longitudinal ribs. Figure 5 shows specimens of the corrugated pipe and the ribbed pipe with the bearing blocks which were used in testing them.

As has been stated, in general three specimens of each size and type of pipe were tested. On one pipe of each group of three, vertical deflections were measured during the loading tests with micrometer dials reading to 0.001 inch. Two such dials were used, one at each

TABLE 1.—*Smooth pipe, sand-cast, manufacturer A, 3-edge bearing, load applied at center of pipe unless otherwise noted*[1]

Type	Identification no.	Nominal size	Weight	Thickness at break[2]	Overall length	Laying length	Length of bearing	Load at failure	Supporting strength, pounds per foot of diameter per foot of laying length		Remarks
		Inches	*Pounds*	*Inches*	*Inches*	*Inches*	*Inches*	*Pounds*		*Average*	
Plain end	SP-12-1	12	172	0.45	36	36	36	25,550	8,520		
	2	12	170	.44	36	36	36	23,250	7,750	7,970	
	3	12	170	.44	36	36	36	22,950	7,650		
Bell end	SB-12-1	12	190	.49	38½	35¾	30	18,750	6,290		Load applied at center of bearing.
	2	12	190	.42	38½	35¾	30	18,950	6,360	6,340	Do.
	3	12	192	.46	38½	35¾	30	18,950	6,360		Do.
Plain end	SP-16-1	16	256	.55	35¾	35¾	35¾	24,250	6,100		
	2	16	250	.50	36	36	36	22,050	5,510	5,730	
	3	16	252	.51	36	36	36	22,310	5,580		
Bell end	SB-16-1	16	300	.54	38¼	35½	30	25,450	6,450		
	2	16	296	.52	38½	35½	30	24,200	6,140	6,490	
	3	16	304	.55	38¼	35¾	30	27,300	6,870		Broke through the bell.
Plain end	SP-20-1	20	346	.53	36¼	36¼	36¼	21,950	4,360		
	2	20	346	.50	35¾	35¾	35¾	19,500	3,930	3,880	
	3	20	314	.50	36	36	36	16,750	3,350		
Bell end	SB-20-1	20	404	.54	38½	35¾	30	25,600	5,150		Do.
	2	20	376	.50	38½	35¾	30	19,250	3,870	4,810	
	3	20	406	.58	38½	36	30	27,050	5,410		Do.
Plain end	SP-24-1	24	416	.54	35¾	35¾	35¾	14,300	2,400		
	2	24	432	.56	35¾	35¾	35¾	18,000	3,020	2,900	Crack in one side before test.
	3	24	424	.55	35¾	35¾	35¾	19,600	3,290		
Bell end	SB-24-1	24	540	.60	39	35¾	30	25,850	4,340		
	2	24	532	.57	39	35¾	30	23,500	3,940	3,980	
	3	24	538	.58	39	35¾	30	21,850	3,670		
Plain end	SP-30-1	30	644	.68	35¾	35¾	35¾	22,300	2,990		
	2	30	662	.74	35¾	35¾	35¾	21,200	2,850	2,860	
	3	30	646	.73	36	36	36	20,500	2,730		
Bell end	SB-30-1	30	770	.79	39	35¾	30	26,300	3,530		Broke through the bell.
	2	30	768	.73	39	35¾	30	22,900	3,070	3,320	Do.
	3	30	784	.75	39	36	30	25,250	3,370		Do.
Plain end	SP-36-1	36	882	.84	35¾	35¾	35¾	25,400	2,840		
	2	36	886	.79	36	36	36	16,450	1,830	3,000	Flaw in one end of break. Not included in average
	3	36	1,000	.91	36	36	36	28,500	3,170		
Bell end	SB-36-1	36	1,070	.87	39	35¾	30	26,850	3,000		Broke through the bell.
	2	36	1,086	.85	39	35¾	30	31,600	3,530	3,330	Do.
	3	36	1,052	.82	39	35¾	30	30,850	3,450		Do.
Plain end	SP-42-1	42	1,300	1.06	36	36	36	35,950	3,420		
	2	42	1,340	1.00	36	36	36	31,300	2,980	3,120	
	3	42	1,230	.93	36	36	36	30,950	2,950		
Bell end	SB-42-1	42	1,520	1.00	39	36	30	39,900	3,800		Do.
	2	42	1,470	.92	39	36	30	36,550	3,480	3,650	Do.
	3	42	1,540	1.02	39	36	30	38,500	3,670		Do.
Plain end	SP-48-1	48	1,400	1.02	36	36	36	25,200	2,100		
	2	48	1,420	.90	36	36	36	22,300	1,860	1,970	
	3	48	1,400	.97	36	36	36	23,450	1,950		
Bell end	SB-48-1	48	1,695	1.00	39	36	30	31,950	2,660		Do.
	2	48	1,712	1.03	39	36	30	29,350	2,450	2,390	Do.
	3	48	1,680	.92	39	36	30	24,850	2,070		Do.

[1] Lower bearing blocks 1 inch apart for 12- and 16-inch pipe; 2 inches for 20-, 24-, and 30-inch pipe; 3 inches for 36-, 42-, and 48-inch pipe.
[2] Plain-end pipe—thickness is average of measurements at 10 points equally spaced along the crack. Bell-end pipe—thickness of barrel at spigot end.

end of the pipe. They were set 1 inch in from the ends of the pipe except when the specimens had bell ends, in which case the dial at the bell end was set 1 inch in from the inside shoulder of the bell. Figure 6 shows the position of the dials as they were set to measure the deflections of smooth pipe with plain ends. Deflections were observed for increments of load up to maximum values of from 10,000 to 15,000 pounds, after which the readings were discontinued. The results of the 3-edge bearing tests are shown in tables 1 to 5, inclusive.

These tables show the principal dimensions of each pipe tested and the load at failure. As a basis for comparison they also show the "supporting strength" in pounds per foot of diameter per foot of laying length. This supporting strength was determined by dividing the applied load by the product of the internal diameter and the laying length (in feet) and is hereafter referred to as the "strength."

LOCATION OF CRACK AT FAILURE AN INDICATION OF UNIFORMITY OF SPECIMEN

When an elastic ring is supported along the bottom element and a concentrated load is applied along the top element, as in the 2-edge bearing test, the maximum positive bending moments in the ring occur at the top and bottom, directly under the load and directly over the support. The bending moment in the ring decreases from a maximum positive value at the crown to a maximum negative value at the horizontal axis passing through zero at a point the radius to which makes an angle with the vertical of approximately 40°.[2] The maximum positive bending moment at the crown is about 75 percent greater than the maximum negative moment at the horizontal axis.

In the 3-edge bearing test, as conducted on cast-iron pipe of ordinary dimensions, the bending moments are

[2] Tests of Cast-Iron and Reinforced Concrete Culvert Pipe, by Arthur N. Talbot, Bulletin No. 22, University of Illinois.

TABLE 2.—*Smooth pipe, sand-cast, manufacturer B, 3-edge bearing, load applied at center of pipe* [1]

Type	Identification no.	Nominal size	Weight	Thickness at break [2]	Over-all length	Laying length	Length of bearing	Load at failure	Supporting strength, pounds per foot of diameter per foot of laying length		Remarks
		Inches	*Pounds*	*Inches*	*Inches*	*Inches*	*Inches*	*Pounds*		*Average*	
Plain end	SP-12-1	12	172	0.51	36	36	36	24,600	8,200		
	2	12	161	.46	36	36	36	20,700	6,900	7,920	
	3	12	164	.48	36	36	36	25,950	8,650		
Bell end	SB-12-1	12	185	.50	39	36¼	35½	29,150	9,650		Broke through the bell
	2	12	183	.42	39	36¼	35½	26,350	8,720	9,270	Do.
	3	12	183	.50	39	36¼	35½	28,500	9,430		Do.
Plain end	SP-18-1	18	226	.48	36	36	36	12,850	2,860		
	2	18	225	.41	36	36	36	14,600	3,240	3,390	
	3	18	235	.47	36	36	36	18,350	4,080		
Bell end	SB-18-1	18	275	.45	39	36	35	21,050	4,680		
	2	18	277	.43	39	36	35	14,750	3,280	3,940	
	3	18	276	.50	39	36	35	17,350	3,860		
Plain end	SP-24-1	24	405	.60	35	35	35	17,050	2,920		
	2	24	434	.62	35	35	35	21,300	3,650	3,340	
	3	24	406	.60	35	35	35	20,100	3,450		
Bell end	SB-24-1	24	474	.62	39	36	35	19,950	3,320		Do.
	2	24	471	.60	39	36	35	19,800	3,300	3,300	Do.
	3	24	464	.56	39	36	35	19,650	3,280		Do.
Plain end	SP-30-1	30	582	.70	34¾	34¾	34¾	19,650	2,710		
	2	30	574	.68	35	35	35	19,350	2,650	2,600	
	3	30	587	.68	35	35	35	17,750	2,430		
Bell end	SB-30-1	30	652	.62	39	36	35	21,650	2,890		
	2	30	665	.50	39	36	35	16,000	2,130	2,530	Do.
	3	30	654	.64	39	36	35	19,350	2,580		
Plain end	SP-36-1	36	[3] 940	.95	34½	34½	34½	30,450	3,530		
	2	36	[3] 940	.94	34½	34½	34½	29,250	3,390	3,460	
	3	36	[3] 940	.91	34½	34½	34½	23,550	[4] 2,730		
Bell end	SB-36-1	36	[3] 1,100	.92	39	36	34½	37,550	4,170		Do.
	2	36	[3] 1,100	.95	39	36	34½	32,100	3,570	3,870	Do.
	3	36	[3] 1,100	.92	39	36	34½	34,950	3,880		Do.

[1] Lower bearing blocks 1 inch apart for 12- and 18-inch pipe; 2 inches for 24- and 30-inch pipe; 3 inches for 36-inch pipe.
[2] Plain-end pipe—thickness is average of measurements at 10 points equally spaced along the crack. Bell-end pipe—thickness of barrel at spigot end.
[3] Approximate.
[4] Cracked before test, not included in average.

TABLE 3.—*Smooth pipe, centrifugally-cast, 3-edge bearing, load applied at center of pipe unless otherwise noted* [1]

Type	Identification no.	Nominal size	Weight	Thickness at break [2]	Over-all length	Laying length	Length of bearing	Load at failure	Supporting strength, pounds per foot of diameter per foot of laying length		Remarks
		Inches	*Pounds*	*Inches*	*Inches*	*Inches*	*Inches*	*Pounds*		*Average*	
Plain end	CP-12-1	12	165	0.42	36	36	36	25,950	8,650		
	2	12	168	.41	36	36	36	27,050	9,020	8,980	
	3	12	166	.42	36	36	36	27,800	9,270		
Bell end	CB-12-1	12	246	.44	40¾	36¼	34	31,475	10,420		
	2	12	250	.43	41	37	34	34,650	11,230	10,820	
	3	12	246	.43	40½	36¼	30	24,700	8,180		Load applied at center of bearing. Not included in average.
Plain end	CP-16-1	16	278	.51	36	36	36	32,300	8,080		
	2	16	280	.49	36	36	36	32,400	8,100	8,670	
	3	16	282	.58	36¼	36¼	36¼	39,650	9,840		
Bell end	CB-16-1	16	404	.51	40½	36	33	38,000	9,500		
	2	16	400	.51	40½	36	33	40,950	10,240	9,930	
	3	16	402	.51	40¾	36¼	33	40,460	10,040		
Plain end	CP-20-1	20	452	.63	36¼	36¼	36¼	32,350	6,430		
	2	20	440	.65	36½	36½	36½	37,950	7,490	6,830	
	3	20	446	.59	36¼	36¼	36¼	33,100	6,570		
Bell end	CB-20-1	20	570	.64	40½	36¼	33	44,500	8,840		
	2	20	572	.60	41	36½	33	40,950	8,080	8,640	
	3	20	580	.66	41	36¼	33	45,250	8,990		

[1] Lower bearing blocks 1 inch apart for 12- and 16-inch pipe; 2 inches apart for 20-inch pipe.
[2] Plain-end pipe—thickness is average of measurements at 10 points equally spaced along the crack. Bell-end pipe—thickness of barrel at spigot end.

substantially the same as in the 2-edge test. In the 3-edge test, as will be shown later, the moment at the crown is less than in the 2-edge test by an insignificant amount. Also in the 3-edge test the moment at the bottom of the pipe is slightly less than at the top.

Therefore, it is to be expected that a pipe which is exactly circular and of uniform thickness will crack, under the ultimate load in the 3-edge bearing test, along the upper element. Failure of the crack to develop in this position is evidence of nonuniformity, either in cross section or quality of metal. A slight nonuniformity may cause the pipe to break at the bottom rather than at the top but a crack along the horizontal axis evidences nonuniformity of considerable magnitude.

About 80 percent of the specimens of smooth sand-cast pipe furnished by manufacturer A broke at the top and the remainder broke at the bottom except one specimen which broke at a flaw near the bottom. Of the smooth sand-cast pipe of manufacturer B, one

TABLE 4.—*Spiral-corrugated pipe, 3-edge bearing, load applied at center of pipe* [1]

Type	Identification no.	Nominal size	Weight	Thickness at break [2]	Over-all length	Laying length	Length of bearing	Load at first crack	Load at failure	Supporting strength, pounds per foot of diameter per foot of laying length— At first crack	Average	At failure	Average
		Inches	*Pounds*	*Inches*	*Inches*	*Inches*	*Inches*	*Pounds*	*Pounds*				
Pipe	M-15-1	15	152	0.27–0.27	38¾	36	38¾	12,700	15,450	3,390		4,120	
	2	15	151	.30–.30	38¾	36	38¾	14,600	15,000	3,890	3,760	4,000	4,010
	3	15	152	.28–.28	38¾	36	38¾	15,050	15,050	4,010		4,010	
Cuff	F-15-1	15	157	.21–.33	38¾	36	38¾	12,900	12,900	3,440		3,440	
	2	15	153	.22–.32	38¾	36	38¾	13,150	14,700	3,510	3,120	3,920	3,630
	3	15	160	.24–.37	38¾	36	38¾	9,050	13,200	2,410		3,520	
Pipe	M-18-1	18	177	.26–.26	38¾	36	38¾	14,250	14,250	3,170		3,170	
	2	18	169	.28–.29	39	36	39	12,500	13,400	2,780	2,860	2,980	2,990
	3	18	168	.21–.28	38¾	36	38¾	11,800	12,650	2,620		2,810	
Cuff	F-18-1	18	192	.20–.30	38¾	36	38¾	12,950	12,950	2,880		2,880	
	2	18	187	.25–.34	38¾	36	38¾	12,000	12,600	2,670	2,680	2,800	2,770
	3	18	190	.22–.30	38¾	36	38¾	11,250	11,900	2,500		2,640	
Pipe	M-24-1	24	286	.33–.33	39½	36	39½	15,800	15,800	2,630		2,630	
	2	24	285	.32–.32	39½	36	39½	11,300	12,700	1,880	2,230	2,120	2,310
	3	24	286	.32–.32	39½	36	39½	13,050	13,150	2,170		2,190	
Cuff	F-24-1	24	285	.32–.33	39½	36	39½	12,100	12,300	2,020		2,050	
	2	24	289	.35–.40	39½	36	39½	15,250	15,400	2,540	2,350	2,570	2,370
	3	24	291	.33–.36	39½	36	39½	15,000	15,000	2,500		2,500	
Pipe	M-30-1	30	421	.38–.43	39½	36	39½	17,350	20,650	2,310		2,750	
	2	30	420	.34–.39	39½	36	39½	20,250	21,700	2,700	2,670	2,890	2,900
	3	30	425	.38–.38	39½	36	39½	22,400	23,050	2,990		3,070	
Cuff	F-30-1	30	467	.45–.55	39½	36	39½	21,550	22,450	2,870		2,990	
	2	30	481	.37–.49	39½	36	39½	19,950	20,950	2,660	2,710	2,790	2,900
	3	30	480	.35–.45	39½	36	39½	19,500	21,800	2,600		2,910	
Pipe	M-36-1	36	557	.43–.45	40¼	36	40¼	21,400	21,800	2,380		2,420	
	2	36	560	.42–.43	40¼	36	40¼	19,150	22,000	2,130	2,260	2,440	2,420
	3	36	557	.43–.44	40	36	40	20,400	21,700	2,270		2,410	
Cuff	F-36-1	36	646	.45–.57	40	36	40	24,300	26,600	2,700		2,960	
	2	36	636	.41–.56	40¼	36	40¼	23,400	27,000	2,600	2,690	3,000	2,930
	3	36	631	.45–.57	40	36	40	25,000	25,500	2,780		2,830	

[1] Lower bearing blocks 1 inch apart for 15- and 18-inch pipe; 2 inches for 24- and 30-inch pipe; 3 inches for 36-inch pipe.
[2] Thickness at break taken under load at end of pipe which cracked first. First value—thickness at bottom of corrugation. Second value—thickness at top of corrugation.

TABLE 5.—*Ribbed pipe, 3-edge bearing, load applied at center of pipe* [1]

Type	Identification no.	Nominal size	Weight	Thickness at break [2]	Distance from bell end to center line of first transverse rib	Distance between transverse ribs	Number of transverse ribs	Over-all length	Laying length	Load at first crack	Load at failure	Supporting strength, pounds per foot of diameter per foot of laying length— At first crack	Average	At failure	Average
		Inches	*Pounds*	*Inches*	*Inches*	*Inches*		*Inches*	*Inches*	*Pounds*	*Pounds*				
Bell end	R-15-1	15	150	0.24–0.79	5⅞	6⅞	5	36	34½	9,850	11,100	2,740		3,090	
	2	15	150	.33–.91	5⅞	6⅞	5	36	34½	11,250	12,900	3,130	2,940	3,590	3,310
Do	R-18-1	18	230	.30–1.05	5⅝	6 1/16	6	38½	37	19,000	22,650	4,110		4,900	
	2	18	228	.28–1.03	5⅝	6 1/16	6	38½	37	19,650	19,650	4,250	4,010	4,250	4,600
	3	18	228	.28–1.00	5⅝	6 1/16	6	38½	37	17,000	21,450	3,680		4,640	
Do	R-24-1	24	310	.25–1.20	6½	7 3/16	5	37½	36	13,300	15,000	2,220		2,500	
	2	24	306	.33–1.20	6½	7 3/16	5	37½	36	14,850	18,550	2,470	2,350	3,090	2,810
	3	24	303	.30–1.16	6½	7 3/16	5	37½	36	14,200	17,000	2,370		2,830	

[1] Lower bearing blocks 1 inch apart for 15- and 18-inch pipe; 2 inches for 24-inch pipe.
[2] First value—thickness of barrel at spigot end. Second value—total thickness of barrel and rib at first rib from spigot end.

third broke at the top and one third broke either at the bottom, or at the top and bottom approximately simultaneously. The remainder, except two in which the exact location of the crack was not determined, broke either at the side, or at the top and side, bottom and side, or top, bottom and side, simultaneously. About 65 percent of the specimens of spiral corrugated pipe broke at the top and the remainder, except three specimens in which the exact location of the crack was not determined, broke at the bottom. All specimens of the smooth, centrifugally cast pipe broke along the top indicating a greater uniformity in this group than in the other three which have been discussed. This is probably due to the method of manufacture which may be expected to result in a more uniform thickness of metal than sand-casting methods.

Ribbed pipe of the type tested is not of uniform cross section, and failure of the shell will not necessarily occur at the top. The bending moment on the horizontal axis through the center of the ring produces tensile stress in the outer fibers and the tensile stress produced in the small rib sections may be sufficient to produce initial failure at this point even though the bending moment is much less than at the crown, where the ribs are in compression. This type of failure actually occurred as was evidenced by crack development during the tests and by the fact that all of the eight specimens tested broke along both sides. Five of

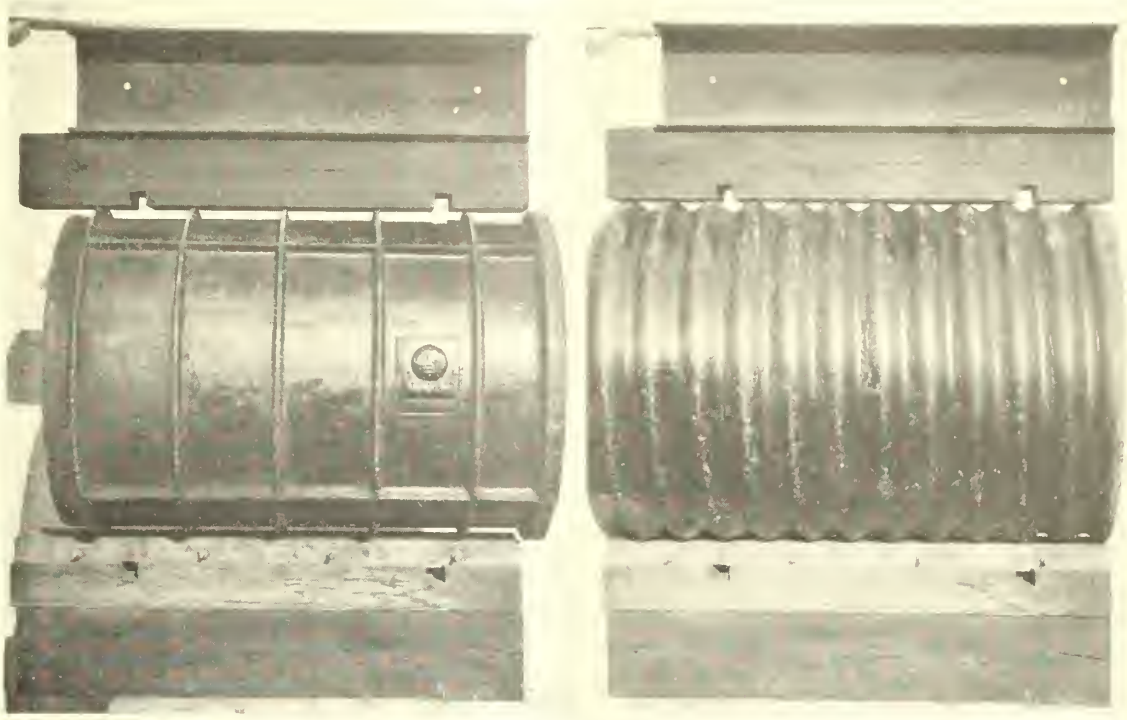

FIGURE 5.—RIBBED PIPE AND SPIRAL-CORRUGATED PIPE WITH BEARING BLOCKS USED IN 3-EDGE BEARING TEST.

them also broke at the top and one of these five also broke at the bottom.

In testing the smooth pipe with bell ends it was observed in numerous cases that, at failure, a crack developed in the body of the pipe without the development of a visible crack in the bell. The other cases, in which the crack developed throughout the length of the specimen, are indicated in tables 1 and 2.

INFLUENCE OF POSITION OF LOAD SHOWN IN TEST RESULTS

The test results for 12-inch pipe shown in tables 1 and 3 show that the position of the center of load application on the upper bearing block has an important influence on the strength developed by pipe with bell ends in the 3-edge bearing test. The tests of 12-inch pipe with bell ends reported in table 1 were all with the load applied at the center of the bearing block. The center of the load was 15 inches from the spigot end and the eccentricity of load, with respect to the center of length of the pipe, was 4¼ inches. The average strength of the bell-end pipe (6,340 D) was only 80 percent of the average strength of the 12-inch pipe with plain ends (7,970 D) on which the load was applied at the center of the pipe. In all the other tests of bell-end pipe shown in table 1 the load was applied at the center of the pipe and in every case the average strength of the bell-end pipe was greater than the average strength of plain-end pipe of corresponding size.

Bell-end pipe CB-12-3 (table 3) was tested with the point of load application at the center of the bearing block, the eccentricity with respect to the center of the pipe being 5¼ inches. The strength of this pipe was only 76 percent of the average strength of specimens CB-12-1 and CB-12-2 which were tested with the load in the center of the pipe. Also, the strength was only 91 percent of the average strength of the 12-inch pipe with plain ends. As in table 1, the average strength of all other bell-end pipe in table 3 is greater than the average strength of plain-end pipe of corresponding size. The influence of the position of the point of load application is also shown by the deflection measurements which will be discussed later.

It is apparent that the application of load at the center of the bearing block rather than at the center of length of the pipe subjects pipe with bell ends to an unduly severe load condition which results in an apparent lowering of their strength in the 3-edge bearing test. That it is not a fair test is evidenced by the failure of pipe loaded in this manner under lower loads than for failure of plain-end pipe of similar dimensions.

STRENGTH TESTS DISCUSSED

The average strength of bell-end specimens of the smooth, sand-cast pipe of manufacturer A (table 1),

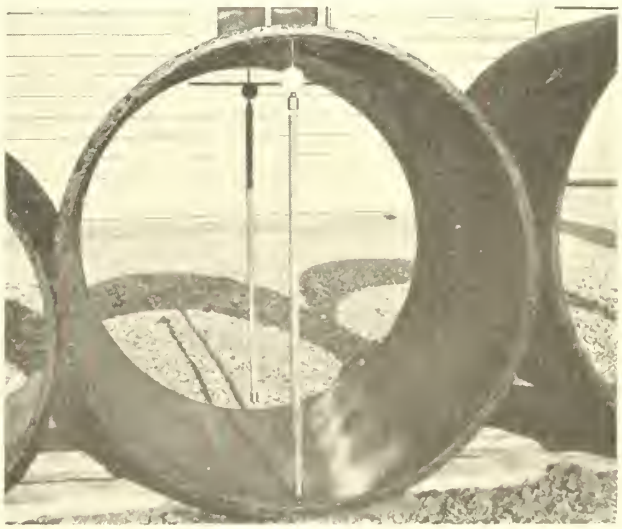

FIGURE 6.—POSITION OF MICROMETER DIALS FOR MEASURING DEFLECTIONS OF SMOOTH PIPE WITH PLAIN ENDS.

TABLE 6.—*Vertical deflections of pipe*

Nominal diameter, inches	Smooth sand-cast pipe, manufacturer A											
	Total load of 3,000 pounds per foot of laying length					Total load, 4,000 pounds per foot of laying length				Plain-end pipe		
	Bell-end pipe			Plain-end pipe		Bell-end pipe			Plain-end pipe			
	Identification no.	Deflections			Identification no.	Deflections[1]	Deflections			Deflections[1]	$\frac{t}{d}$	$\frac{\left(1+\frac{t}{d}\right)^3}{\left(\frac{t}{d}\right)^3}$
		Bell	Spigot	Average			Bell	Spigot	Average			
		Inches	*Inches*	*Inches*		*Inches*	*Inches*	*Inches*	*Inches*	*Inches*		
12	SB-12-3	0.013	0.121	[2] 0.067	SP-12-1	0.078	0.019	0.173	[2] 0.096	0.111	0.0369	22,200
16	SB-16-3	.038	.115	.076	SP-16-1	.116	.055	.161	.108	.165	.0340	28,100
20	SB-20-1	.066	.193	.130	SP-20-2	.198	.091	.270	.180	.285	.0244	74,000
24	SB-24-3	.081	.248	.164	SP-24-1	.344	.113	.341	.228	.497	.0220	100,200
30	SB-30-2	.119	.265	.192	SP-30-1	.316	.167	.371	.269	.454	.0225	93,900
36	SB-36-3	.110	.258	.184	SP-36-1	.343	.153	.356	.254	----------	.0232	85,800
42	SB-42-3	.127	.224	.176	SP-42-1	.307	.176	.309	.242	.430	.0252	67,300
48	SB-48-1	.196	.375	.286	SP-48-1	[3] .630	.272	.520	.396	----------	.0211	113,400
	Smooth sand-cast pipe, manufacturer B											
12	SB-12-2	0.033	----------	----------	SP-12-3	0.066	0.047	----------	----------	0.094	0.0400	17,600
18	SB-18-3	.058	.314	.186	SP-18-2	.316	.081	0.437	0.259	.450	.0244	74,000
24	SB-24-2	.101	.252	.178	SP-24-2	.217	.144	.350	.247	.306	.0258	62,900
30	SB-30-2	.151	.417	.284	SP-30-2	.355	.212	.586	.399	.499	.0227	91,500
36	SB-36-2	.075	.214	.144	SP-36-1	.251	.104	.293	.198	.346	.0264	58,800
	Smooth centrifugally cast pipe											
12	CB-12-2	0.016	0.073	0.044	CP-12-2	0.090	0.023	0.102	0.062	0.123	0.0331	30,400
16	CB-16-1	.034	.100	.067	CP-16-1	.098	.048	.139	.094	.138	.0311	36,400
20	CB-20-3	.036	.108	.072	CP-20-2	.103	.049	.146	.098	.143	.0322	32,900

Nominal diameter, inches	Spiral-corrugated pipe						Ribbed pipe						
	Total load, 3,000 pounds per foot of laying length				Total load, 2,000 pounds per foot of laying length		Identification no.	Total load, 3,000 pounds per foot of laying length			Total load, 2,000 pounds per foot of laying length		
	Identification no.		Deflections[1]		Deflections[1]			Deflections			Deflections		
	Pipe	Cuff	Pipe	Cuff	Pipe	Cuff		Bell	Spigot	Average	Bell	Spigot	Average
15	M-15-1	F-15-2	0.088	0.092	0.053	0.056	R-15-1	0.105	0.200	0.152	0.068	0.126	0.097
18	M-18-2	F-18-3	.168	[3] .168	.105	.103	R-18-2	.088	.130	.109	.056	.081	.068
24	M-24-3	F-24-3	.190	.186	.118	.116	R-24-3	.169	.215	.192	.108	.138	.123
30	M-30-2	F-30-3	.272	.237	.174	.154	----------	----------	----------	----------	----------	----------	----------
36	M-36-2	F-36-1	.327	.275	.212	.176	----------	----------	----------	----------	----------	----------	----------

[1] Average of deflections measured at each end of specimen. [2] Load applied at center of upper bearing block. [3] Extrapolated.

and the smooth, centrifugally cast pipe (table 3) which were loaded at the center of length was greater in all cases than the average strength of the corresponding specimens with plain ends. However, this was not always true of individual specimens.

For example: Bell-end pipe SB-20-2 developed a lower strength than the corresponding plain-end pipes SP-20-1 and SP-20-2; pipe SB-36-1 was not as strong as the plain-end specimen SP-36-3; and specimen SB-48-3 showed a lower strength than specimen SP-48-1. In the centrifugally cast pipe, bell-end specimen CB-16-1 developed a lower strength than the corresponding plain-end specimen CP-16-3. These apparent discrepancies may be attributed partly to differences in quality of metal and partly to differences in shell thickness due to unavoidable inaccuracies in manufacture.

In the smooth, sand-cast pipe of manufacturer B (table 2) the effect of the bell ends on strength is less pronounced than in the pipe of manufacturer A and the centrifugally cast pipe. This is probably due to differences in the design of the bells. (See fig. 7 for typical cross sections of 12-inch pipe.) There are more cases in which individual specimens with bell ends developed less strength than specimens of the same size with plain ends. In the case of the 24- and 30-inch pipe, even the average strength of the bell-end pipe was somewhat less than that of the pipe with plain ends.

Pipe with bell ends normally may be expected to develop a greater strength in the 3-edge bearing test, with the load applied at the center of length of the pipe, than pipe of the same size and dimensions with plain ends. However, these tests have shown that this differential in strength cannot always be expected on account of inaccuracy in manufacture and differences in quality of metal. It is indicated also that the amount of any strength differential which exists is influenced by the design of the bell. It is reasonable to assume that the differential is also dependent on the length of the specimen, being less for long pipe than for short ones. Since culvert pipe is furnished in lengths up to 12 feet, and it has been found in these tests that the 3-foot

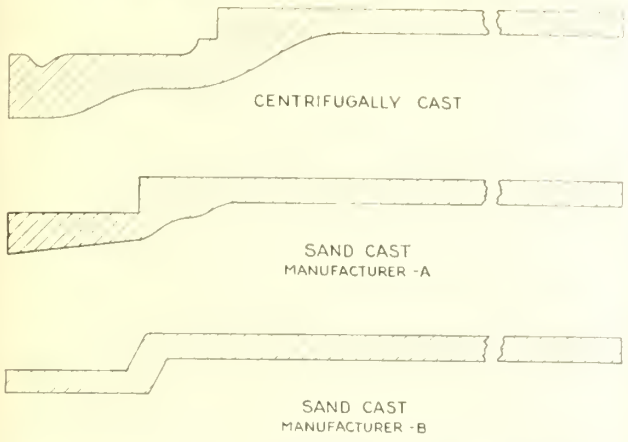

FIGURE 7.—SECTIONS OF BELL-END PIPE, NOMINAL DIAMETER 12 INCHES.

specimens with bell ends did not consistently show a higher strength than pipe with plain ends, it is concluded that the design of cast-iron pipe to withstand the 3-edge bearing test may more safely be based on the results of tests on plain-end specimens than on tests of bell-end specimens.

In the tests of spiral corrugated pipe (table 4) and ribbed pipe (table 5) it was observed that the specimens cracked at a load which was generally, but not always, somewhat less than the ultimate load. Both the load at which the first crack developed and the ultimate load are reported although the former is considered to be more significant of the ability of the pipe to withstand the strength test.

The average strength at first crack of all sizes of corrugated pipe and ribbed pipe exceeded 2,000 pounds per foot of diameter per foot of length and only one specimen (M-24-2) developed a strength lower than this.

The supporting strength of spiral corrugated pipe is so dependent on the dimensions of the corrugations and that of ribbed pipe is so dependent on the size and location of the ribs that no general conclusions may be drawn from the results of these tests. The strengths reported are those developed by pipe of certain dimensions and design and it is not possible to derive from these results data applicable to pipe of different design or dimensions. The only conclusion which appears to be justified is that corrugated or ribbed pipe, similar in all respects to those included in these tests, may be expected to develop a strength in the 3-edge bearing test of at least $2{,}000D$ pounds per linear foot of laying length.

DEFLECTION OF PIPE DISCUSSED

Typical load-deflection curves obtained in these tests are shown in figure 8. The curves for all the other specimens are of the same general character varying only in the magnitude and relationship of deflections at a given load.

The significant data with respect to vertical deflections are given in table 6. This table shows the deflections of all specimens under a load of 3,000 pounds per foot of laying length, the maximum load at which readings on all pipe were available. For comparison, a second set of deflections under a different load is also given. For the smooth sand-cast and centrifugally cast pipe this load is 4,000 pounds per foot of laying length and for the spiral-corrugated and ribbed pipe it is 2,000 pounds per foot of laying length.

The smooth sand-cast pipe, SB-12-3, of manufacturer A was the only bell-end specimen on which deflection readings were obtained with the load applied at the center of the bearing block rather than at the center of the length of the pipe. Comment has already been made regarding the effect of this eccentric load application on the supporting strength and, as is to be expected, its influence on deflections is apparent. Comparing the deflections of specimen SB-12-3 under a load of either 3,000 or 4,000 pounds it will be noted that the deflection of the spigot end is about 56 percent greater than the deflection of the corresponding plain-end specimen, SP-12-1. For the 16- and 20-inch pipe of manufacturer A and the 18-inch pipe of manufacturer B, all of which were tested with the load applied at the center of length of the pipe, the deflections of the spigot end of the bell-end pipe are about the same as the deflections of the corresponding pipes with plain ends. Other things being equal, deflections are directly proportional to loads and therefore it is to be expected, as has already been shown to have been the case, that an eccentricity of load application which causes the spigot end of a bell-end pipe to deflect much more than a plain-end pipe of the same metal and dimensions under symmetrical loading, will cause the bell-end pipe to fail at a lower load than the pipe with plain ends.

The data of table 6 and the curves of figure 8, which are all concave downward, show that the deflections increased at a somewhat greater rate than the corresponding loads.

A comparison of the average deflections of bell-end pipe with those of pipe with plain ends shows that in all cases, the presence of the bell resulted in a greater stiffness of pipe. However, only short lengths of pipe were tested. The increased stiffness is not always the same and this may be attributed to differences in the bells themselves.

Comparison of deflections of different types of pipe of the same size shows some differences in flexibility but it should not be concluded that these differences are necessarily inherent in the type itself. The type may have a considerable influence, as in the case of the spiral-corrugated pipe, which naturally is much stiffer than a smooth pipe of the same shell thickness, but the details of design are also of great importance. In general, pipe of the larger sizes show a greater flexibility than those of smaller size but this need not necessarily be the case. It should be remembered that deflection of a smooth pipe under a given load is dependent on three variables—diameter, shell thickness, and the modulus of elasticity of the metal.

The vertical deflection of smooth pipe with plain ends in the 2-edge bearing test [3] is given by the equation [4]—

$$\Delta y = 0.15 \frac{Pr^3}{EI} \qquad (2)$$

where

Δy = vertical deflection, in inches,
P = load, in pounds,
r = radius of center line of shell, in inches,
E = modulus of elasticity, in pounds per square inch,
I = moment of inertia of a longitudinal element of the shell, in inches[4].

[3] As will be shown in the discussion of modulus of rupture of pipe, this test may be considered, for all practical purposes, to be the same as the 3-edge bearing test.
[4] Tests of Cast-Iron and Reinforced Concrete Culvert Pipe, by Arthur N. Talbot, Bulletin No. 22, University of Illinois.

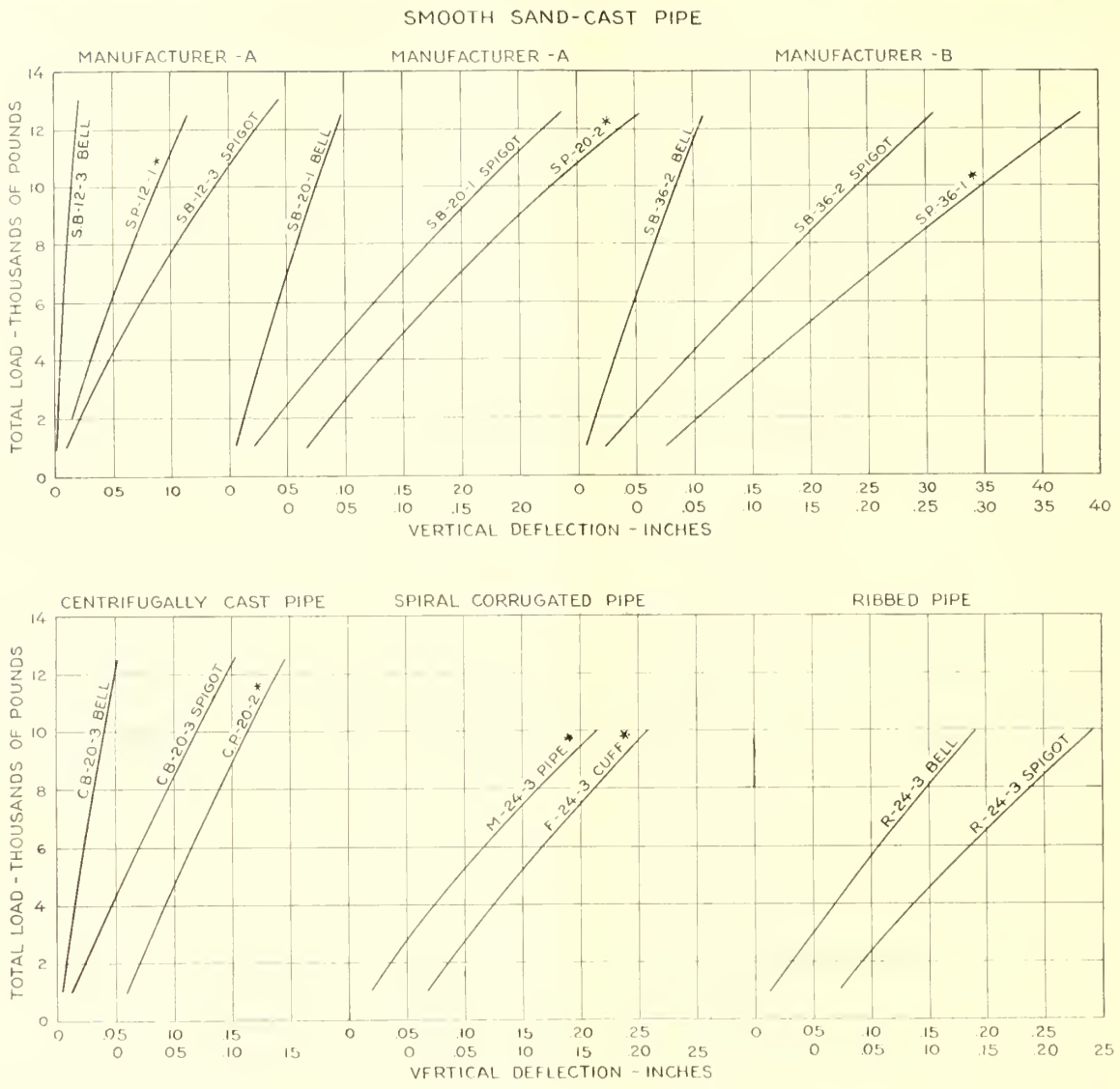

FIGURE 8.—TYPICAL LOAD-DEFLECTION CURVES. AN ASTERISK FOLLOWING THE SERIAL NUMBER INDICATES DEFLECTIONS ARE AVERAGES OF MEASUREMENTS AT EACH END OF PIPE.

This equation is applicable to pipe of constant section and constant modulus of elasticity. Cast-iron pipes, due to unavoidable inaccuracies in manufacture, are not accurately uniform in cross section and the modulus of elasticity of cast iron is not a constant. However, the formula is useful as a basis of comparison and, as has been pointed out by Talbot, may be used somewhat empirically.

By making suitable substitutions, equation 1 may be reduced to the form—

$$\Delta y = \frac{0.225P\left(1+\frac{t}{d}\right)^3}{Eb\left(\frac{t}{d}\right)^3} \qquad (2)$$

where

t = shell thickness, in inches,
d = inside diameter of pipe, in inches,
b = length of pipe, in inches.

For convenience equation 2 may be written—

$$\Delta y = \frac{0.225P}{Eb} \qquad (3)$$

where

$$K = \frac{\left(1+\frac{t}{d}\right)^3}{\left(\frac{t}{d}\right)^3}$$

Thus, for constant value of load, modulus of elasticity, and length of pipe, there is a straight-line relationship between deflections and values of K.

Values of $\frac{t}{d}$ and K for the smooth pipe with plain ends are given in table 6 and, in figure 9, these values of K have been plotted against the corresponding observed deflections. The values of t used in computing K are the average thicknesses as measured along the crack in the pipe and given both in tables 1, 2, and 3 and tables 7, 8, and 9. Values of d are the average inside diameters as given in tables 7, 8, and 9. Since the pipe shells did not have a constant thickness and were not always exactly circular in cross section, these values of t and d are not necessarily the true values which should be used in computing deflections. However,

they are the best figures available and it is believed that there are no departures from true values of sufficient magnitude to introduce errors of consequence in this discussion. Therefore, the deviations of the plotted points from the straight-line relationships shown in figure 9 are due primarily to deviations of the moduli of elasticity from the average modulus at this particular load.

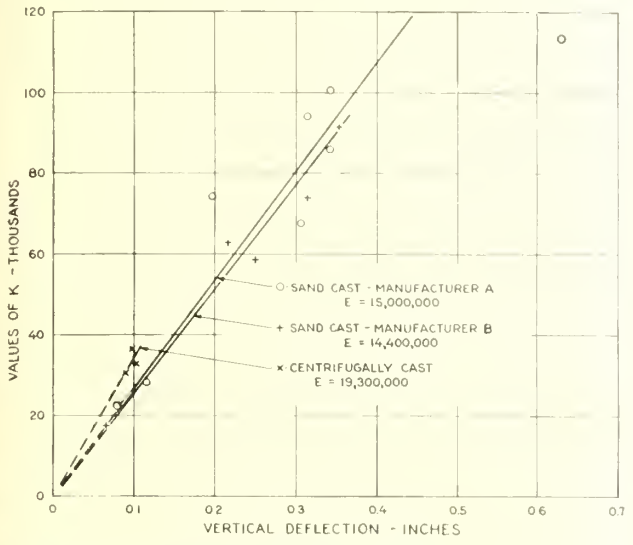

FIGURE 9.—RELATIONSHIP BETWEEN VERTICAL DEFLECTIONS IN 3-EDGE BEARING TEST AND VALUES OF K FOR SMOOTH CAST-IRON PIPE WITH PLAIN ENDS BASED ON A LOAD OF 3,000 POUNDS PER LINEAR FOOT OF PIPE.

The average straight-line relationships between vertical deflections and values of K, for the three groups of pipe, correspond to moduli of elasticity of 14,400,000, 15,000,000, and 19,300,000 pounds per square inch, respectively, for the sand-cast pipe of manufacturer B, the sand-cast pipe of manufacturer A and the centrifugally cast pipe. This order of values of moduli is not in agreement with the order of values shown by the strip tests which will be described later. The average values of moduli of elasticity, as determined by tests on strips, are lowest for the sand-cast pipe of manufacturer A and highest for the centrifugally cast pipe.

While there is a fairly large difference between the average moduli of the sand-cast pipe and that of the centrifugally cast pipe, it is apparent that, for the smooth pipe included in this study, the magnitude of the vertical deflections under a given load in the 3-edge bearing test was influenced more by variations in the ratio of shell thickness to diameter than by variations in the modulus of elasticity of the metal.

MODULUS OF RUPTURE OF PIPE STUDIED

The load capacity of cast-iron pipe in the 3-edge bearing tests is influenced by the diameter of the pipe, the thickness of the shell, and the strength of the metal, and it is impossible to study the influence of any one of these variables without a knowledge of the other two. In any given case, the first two variables may be determined by measurement. As a measure of the third variable, the strength of the metal, the modulus of rupture, computed from the results of tests, is useful.

The 2-edge or "knife-edge" bearing test is the same as the 3-edge bearing test except that in the former the lower bearing consists of a single strip instead of two strips as in the latter. Thus, in the 2-edge test the specimen is loaded through the upper and lower elements which are in a vertical plane through the longitudinal axis of the pipe. It has been shown [5] that in a thin elastic ring loaded in this manner the maximum moment occurs under the load and is given by the equation

$$M_2 = \frac{Pr}{\pi} \tag{4}$$

where

M_2 = bending moment, in inch-pounds,
P = load, in pounds,
r = radius of center line of ring, in inches.

It has also been shown [5] that, for a thin, elastic ring in the 3-edge bearing test, the moment under the load is given by the equation

$$M_3 = \frac{Pr}{2\pi}\left(1 + \sqrt{1 - \frac{a^2}{r^2}} + \frac{a}{r}\alpha - \frac{a^2}{r^2}\right) \tag{5}$$

where

M_3 = bending moment in inch-pounds,
P = load in pounds,
r = radius of center line of ring in inches,
a = one half the distance, between the lower bearing strips in inches,
α = angle, in radians, whose sine equals $\frac{a}{r}$.

Equation 4 gives calculated bending moments somewhat larger than those given by equation 5 but the difference is so slight as to be negligible for pipe having the dimensions of those which are under discussion. For example, for a pipe having an inside diameter of 12 inches and a shell thickness of one half inch, the moment for the 2-edge bearing exceeds that for the 3-edge bearing with lower bearings 1 inch apart, by less than 1 percent. The same is true for a pipe having an inside diameter of 48 inches, a shell thickness of 1 inch, and the lower bearings in the 3-edge test spaced 4 inches apart. Since this is the case, it will be sufficiently accurate, and more convenient on account of its greater simplicity, to use the equation for the 2-edge bearing test in calculating the modulus of rupture for pipe tested in the 3-edge test.

The modulus of rupture may be calculated from the usual flexure formula—

$$R = \frac{Mc}{I} \tag{6}$$

where

R = modulus of rupture, or computed fiber stress in pounds per square inch,
M = bending moment in inch-pounds,
c = distance from neutral axis to extreme fiber in inches,
I = moment of inertia of the section (longitudinal element of the shell), in inches^4.

Combining equations 4 and 6,

$$R = \frac{Prc}{\pi I}$$

[5] Tests of Cast-Iron and Reinforced Concrete Culvert Pipe, by Arthur N. Talbot, Bulletin No. 22, University of Illinois. Experimental and Mathematical Analyses of Drain Tile Testing and New Test Bearing, by Dalton G. Miller and Joseph A. Wise, Technical Bulletin 52, University of Minnesota Agricultural Experiment Station.

Let

d = inside diameter of pipe in inches,
t = shell thickness in inches,
b = length of pipe in inches.

Then

$$R = \frac{0.955\,P(d+t)}{bt^2} \qquad (7)$$

When

$b = 12$ inches and P = load in pounds per linear foot of pipe,

$$R = \frac{.0796P(d+t)}{t^2} \qquad (8)$$

The flexure formula, equation 6, is based on the assumptions that the modulus of elasticity is a constant and that the fiber stress is within the elastic limit of the material. For cast-iron at the breaking load these assumptions are not fulfilled and therefore the modulus of rupture, as given by equations 7 or 8, is not the true stress in the outer fiber. However, as with other materials, the calculated modulus of rupture of cast-iron is a useful measure of the strength of the material at its ultimate load.

DESIGN DATA PRESENTED

Values for the moduli of rupture of the smooth pipe with plain ends (described in tables 1, 2, and 3) have been calculated from equation 8 and the results are given in tables 7, 8, and 9. These tables also give the average inside diameters of the pipe, which generally vary somewhat from the nominal diameters; and the maximum and minimum, as well as the average, thickness of shell at the break. In the computations, the average inside diameter was substituted for d and the average shell thickness at the break for t.

TABLE 7.—*Breaking load and moduli of rupture of smooth sand-cast pipe with plain ends of manufacturer A*

Identification no.	Nominal diameter	Average inside diameter	Shell thickness at break			Breaking load	Modulus of rupture	Average	Deviation from grand average
			Maximum	Minimum	Average [1]				
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Lb. per foot*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Per cent*
SP-12-1	12	12.2	0.47	0.44	0.45	8,520	42,300	40,800	13.1
2	12	12.2	.46	.41	.44	7,750	40,300		7.8
3	12	12.2	.46	.41	.44	7,650	39,700		6.1
SP-16-1	16	16.2	.58	.52	.55	8,140	35,900	37,700	4.0
2	16	16.2	.52	.48	.50	7,350	39,100		4.5
3	16	16.2	.54	.49	.51	7,440	38,000		1.6
SP-20-1	20	20.5	.54	.52	.53	7,270	43,300	41,400	15.8
2	20	20.5	.52	.47	.50	6,550	43,700		16.8
3	20	20.5	.51	.50	.50	5,580	37,300		.3
SP-24-1	24	24.5	.55	.53	.54	4,800	32,800	38,200	12.3
2	24	24.5	.59	.53	.56	6,040	38,400		2.7
3	24	24.5	.57	.53	.55	6,580	43,400		16.0
SP-30-1	30	30.2	.71	.65	.68	7,490	39,800	34,400	6.4
2	30	30.2	.77	.72	.74	7,120	32,000		14.4
3	30	30.1	.76	.71	.73	6,830	31,500		15.8
SP-36-1	36	36.2	.85	.82	.84	8,530	35,600	34,600	4.8
3	36	36.0	.93	.88	.91	9,500	33,700		9.9
SP-42-1	42	42.0	1.08	1.04	1.06	11,980	36,500	37,700	2.4
2	42	42.0	1.01	.99	1.00	10,430	35,700		4.5
3	42	42.1	.96	.92	.93	10,320	40,800		9.1
SP-48-1	48	48.4	1.05	1.00	1.02	8,400	31,800	33,500	15.0
2	48	48.5	.93	.87	.90	7,430	36,100		3.5
3	48	48.5	.99	.95	.97	7,820	32,700		12.6
Grand av							37,400		8.7

[1] Average of thickness at 10 equally spaced points along the crack.

TABLE 8.—*Breaking load and moduli of rupture of smooth sand-cast pipe with plain ends of manufacturer B*

Identification no.	Nominal diameter	Average inside diameter	Shell thickness at break			Breaking load	Modulus of rupture	Average	Deviation from grand average
			Maximum	Minimum	Average [1]				
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Lb. per foot*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Per cent*
SP-12-1	12	11.9	0.59	0.44	0.51	8,200	31,100	33,500	9.3
2	12	11.9	.50	.42	.46	6,900	32,100		6.4
3	12	12.0	.51	.44	.48	8,650	37,300		8.7
SP-18-1	18	18.0	.55	.43	.48	4,280	27,300	34,900	20.4
2	18	18.0	.47	.42	.44	4,870	36,900		7.6
3	18	17.9	.52	.44	.47	6,120	40,500		18.1
SP-24-1	24	23.8	.63	.55	.60	5,850	31,500	35,400	8.2
2	24	24.0	.64	.57	.62	7,300	37,200		8.5
3	24	24.0	.61	.60	.60	6,890	37,500		9.3
SP-30-1	30	30.0	.75	.59	.70	6,780	33,800	33,600	1.5
2	30	30.0	.70	.65	.68	6,630	35,000		2.0
3	30	30.0	.72	.59	.68	6,090	32,100		6.4
SP-36-1	36	36.0	1.03	.88	.95	10,590	34,500	34,200	.6
2	36	36.0	.99	.82	.94	10,170	33,800		1.5
Grand av							34,300		7.7

[1] Average of thickness at 10 equally spaced points along the crack.

TABLE 9.—*Breaking load and moduli of rupture of smooth centrifugally cast pipe with plain ends*

Identification no.	Nominal diameter	Average inside diameter	Shell thickness at break			Breaking load	Modulus of rupture	Average	Deviation from grand average
			Maximum	Minimum	Average [1]				
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Lb. per foot*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*
CP-12-1	12	12.4	0.43	0.42	0.42	8,650	50,000	52,800	4.4
2	12	12.4	.42	.40	.41	9,020	54,700		4.6
3	12	12.4	.43	.41	.42	9,270	53,600		2.5
CP-16-1	16	16.4	.53	.50	.51	10,770	55,700	55,600	6.5
2	16	16.2	.50	.49	.49	10,800	59,700		14.1
3	16	16.0	.65	.52	.58	13,130	51,500		1.5
CP-20-1	20	20.2	.63	.62	.63	10,710	44,700	48,600	14.5
2	20	20.2	.68	.63	.65	12,480	49,000		6.3
3	20	20.2	.61	.57	.59	10,960	52,100		0.4
Grand av							52,300		6.1

[1] Average of thickness at 10 equally spaced points along the crack.

In comparing the average moduli of rupture of the three groups of specimens shown in these tables, the relatively high modulus of the centrifugally cast pipe is noteworthy. The pipe of this group have an average modulus of rupture of 52,300 pounds per square inch as compared with 37,400 pounds per square inch for the sand-cast pipe of manufacturer A and 34,300 pounds per square inch for the sand-cast pipe of manufacturer B. These results, as well as those of the strip tests to be described later, show that the centrifugally cast metal in these specimens is unquestionably of materially higher strength than the metal in the specimens which were cast in sand molds in accordance with common foundry practice.

The consistency of the values of modulus of rupture developed by individual specimens in each of the three groups is considered to be reasonably good, the average deviations being 8.7 percent, 7.7 percent, and 6.1 percent, respectively, for the sand-cast pipe of manufacturer A, the sand-cast pipe of manufacturer B, and the centrifugally cast pipe. The relatively low average deviation for the centrifugally cast pipe may be indicative of a more uniform product but this figure is established by fewer specimens than in the other groups and the maximum deviations for individual specimens of centrifugally cast pipe are not greatly lower than are those for individual specimens of the sand-cast pipe of manufacturer A.

It is important to note that, in spite of the reasonably low average deviations, the calculated moduli of rupture of individual specimens vary through a rather wide range in all three groups. For the sand-cast pipe of manufacturer A the modulus of rupture varies from a minimum value of 31,500 pounds per square inch to a maximum of 43,700 pounds per square inch and the maximum deviation from the average is 16.8 percent. The variation for the sand-cast pipe of manufacture B is from 27,300 to 40,500 pounds per square inch with a maximum deviation of 20.4 percent, while for the centrifugally cast pipe the range is from 44,700 to 59,700 pounds per square inch with a maximum deviation of 14.5 percent.

The moduli of the sand-cast pipe of manufacturer A and of the centrifugally cast pipe show a general trend, though not a very marked or consistent one, toward lower values with increasing thicknesses of shell. The moduli developed by the sand-cast pipe of manufacturer B appear to be independent of shell thickness.

A knowledge of the modulus of rupture of cast-iron pipe, which may be expected in any particular case, is useful in designing pipe to resist a given load in the 3-edge bearing test, or in predicting the approximate load which pipe of a given thickness and known quality of metal will support.

The strength of pipe in the 3-edge test is commonly expressed in pounds per linear foot of pipe per foot of pipe diameter. Thus, pipe is said to have a strength of 2,000 D pounds per linear foot, D being the nominal inside diameter of the pipe in feet. In this connection it is convenient to transpose equation 8 into another form.

In this equation—

P = load in pounds per linear foot of pipe,
d = inside diameter of pipe, in inches,
Let L = load in pounds per linear foot of pipe per foot of pipe diameter.

Then $P = \frac{Ld}{12}$

Substituting in equation 8,

$$R = \frac{.0796\ Ld\ (d+t)}{12t^2}$$

and

$$L = \frac{12\ Rt^2}{.0796\ d\ (d+t)} = \frac{12\ R\frac{t^2}{d^2}}{.0796\left(1+\frac{t}{d}\right)} \qquad (9)$$

Thus, for any given value of load, L, and modulus of rupture, R, the ratio of shell thickness to diameter $\frac{t}{d}$ is a constant.

The curves of figure 10 are based on calculations made with equation 9 and may be used to determine the shell thickness of smooth cast-iron pipe for any required strength and assumed modulus of rupture, or to predict the approximate strength of pipe when the ratio, $\frac{t}{d}$, and the approximate modulus of rupture is known.

In designing pipe for strength by the use of equation 9 consideration should be given, in the selection of the value of R to be used, to the range in values of modulus of rupture which may be expected in pipe manufactured to meet the design requirements. Also, some allowance in shell thickness should be made as a safeguard

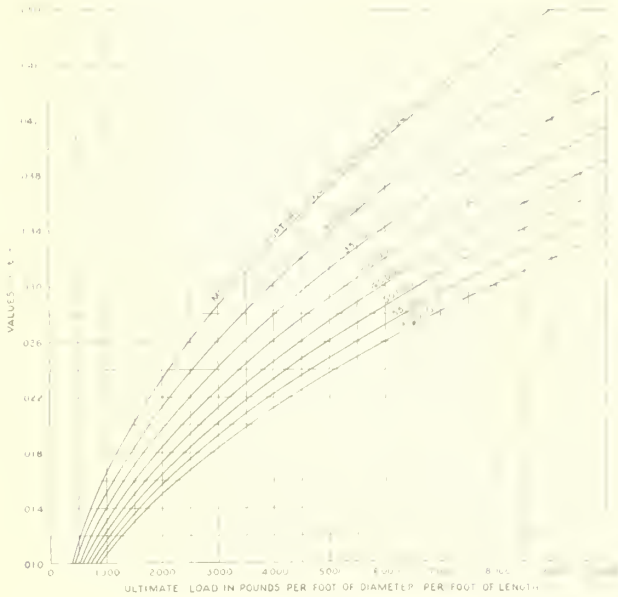

FIGURE 10.—THEORETICAL RELATION BETWEEN ULTIMATE LOAD IN THE 3-EDGE BEARING TEST AND THE RATIO OF SHELL THICKNESS TO INSIDE DIAMETER FOR SMOOTH CAST-IRON PIPE WITH PLAIN ENDS.

against unavoidable inaccuracies in the manufacture of the pipe. An arbitrary increase in the theoretical thickness given by equation 9, of not less than 10 percent is suggested.

BENDING TESTS MADE ON STRIPS CUT FROM PIPE

Some specifications for cast-iron pipe, notably those of the Federal Government, require bending tests on small machined strips cut from sample pipe, and provide limiting values for the modulus of rupture of such strips and for the secant modulus of elasticity at the breaking load.

For example, the following requirements are quoted from Federal Specification WW-P-421 for Cast-Iron Water Pipe (Bell and Spigot):

F-4a. From each 300 lengths of pipe, or fraction thereof, of each size in the contract or order, one length of pipe shall be selected by the inspector before coating. From each sample pipe there shall be cut and machined one test strip 12 inches long, 0.50 inch deep, and the full thickness of the shell in width. This shall be tested as a beam (with machined surfaces top and bottom) on supports 10 inches apart with load applied at two points 3½ inches from the supports. The strip shall be accurately calipered at point of rupture and stress calculated by the formula

$$S = \frac{PLc}{6I}, \text{ or for the above specimen } S = \frac{10P}{b}$$

F-4b. The secant modulus of elasticity at the breaking load shall be calculated by the formula

$$E = \frac{23PL^3}{1296Iy}, \text{ or for the above specimen } E = S\frac{42.6}{y}$$

In these formulas,
S = modulus of rupture,
E = modulus of elasticity,
P = total load,
L = length of span,
c = distance to extreme fiber,
I = moment of inertia,
b = width of specimen (thickness of pipe),
d = depth of specimen,
y = center deflection at load P.

It has been suggested that the modulus of rupture developed by these small test specimens might serve as an index of the modulus of rupture developed by

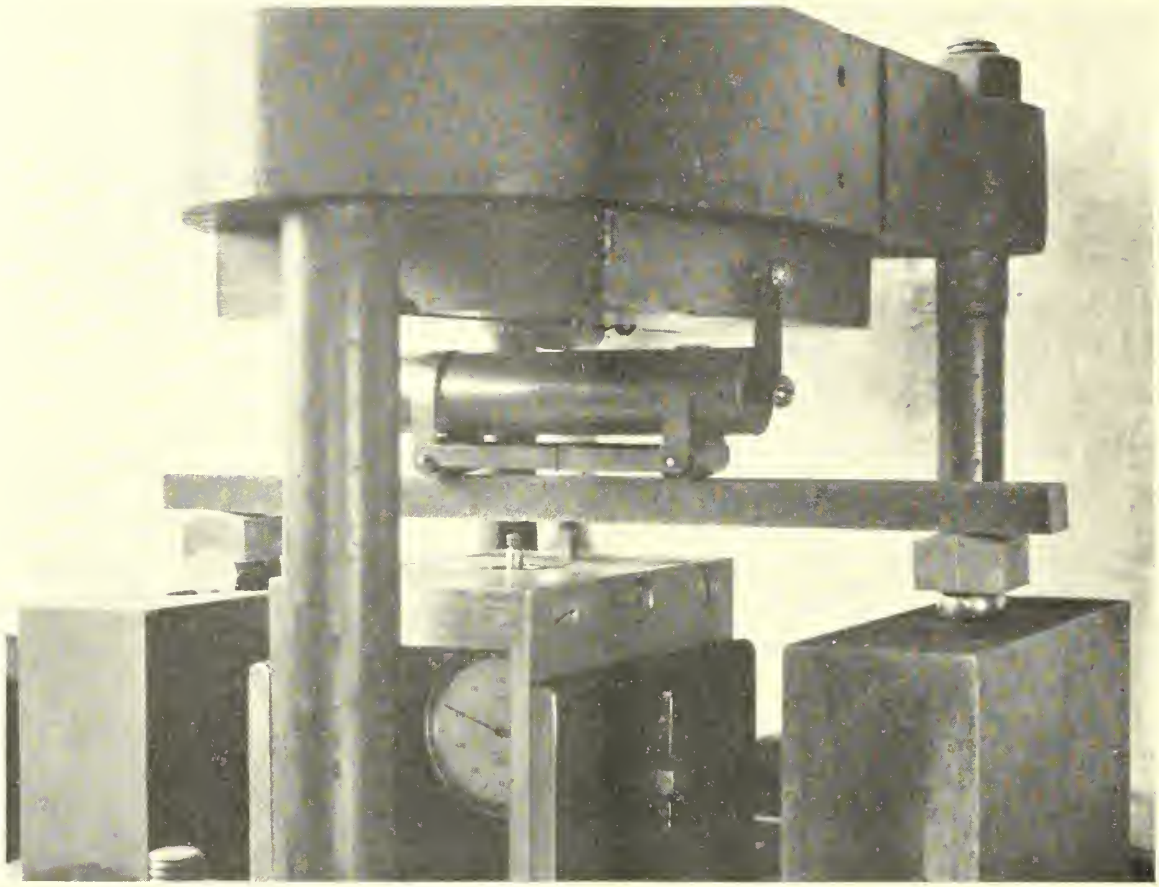

FIGURE 11.—APPARATUS FOR TESTING SMALL STRIPS OF METAL CUT FROM CAST-IRON PIPE.

the specimens of pipe from which the strips are cut in the 3-edge bearing test. The advantages to be gained by testing small strips of metal rather than full-size specimens of pipe, both from the standpoint of convenience and economy, are manifest. In order to determine the suitability of the strip test as a measure of the supporting strength of pipe, a complete series of tests was made on strips cut from all the smooth pipe with plain ends which are listed in tables 7, 8, and 9.

The strips were taken from the pipe which had been broken in the strength tests and were machined and tested in accordance with the specifications quoted above. A 20,000-pound universal testing machine, converted to 2,000-pound capacity for these tests, was used for applying the loads, the moving head being operated at an idle crosshead speed of 0.03 inch per minute.

The loads were applied to the strips at the third points through a system of cylindrical rollers and the specimen was supported by blocks with line-bearings which in turn were supported by a cylindrical bearing at one end and a spherical bearing at the other. A view of the testing apparatus is shown in figure 11.

The cross-sectional dimensions of the specimen at the point of rupture were determined by measurements with micrometer calipers. The modulus of rupture was computed from these dimensions and the load at failure. The pitch coating and any sand adhering to the surface of the metal was removed from the strips by grinding before they were tested.

The deflections of each strip at the center of span, for increments of load up to failure, were measured with a micrometer dial reading to 0.001 inch. The deflection data were used to compute the tangent modulus of elasticity at the origin and the secant modulus of elasticity at failure.

The most convenient location in the broken pipe from which to obtain material for the strips was near the crack which had resulted in the failure of the pipe in the loading test. Since the metal in this area had been subjected to high stress it was necessary to make a preliminary investigation to determine if the physical characteristics of the metal had been materially altered by the stress to which it had been subjected. In this connection it should be noted that the direction of bending in the tests of the pipe was at right angles to that in the bending tests on the strips.

For the preliminary investigation, 3 strips were cut from each of 5 locations in the half circumference of the shell of one of the 48-inch pipes with plain ends of manufacturer A (specimen SP-48-1). The strips were taken from the top (about 2 inches from the crack) and bottom of the shell, from a location on the horizontal axis through the center line and from points midway between this and the top and bottom. Thus there were specimens from two areas of maximum positive bending moment, one area of maximum negative moment, and two areas where the bending moment was very small.

The moduli of elasticity and of rupture calculated from the results of flexure tests on these strips are given in table 10. The results indicated that three strips, one from each of three locations, were defective and the values for these were not included in the calculations of average values. The individual values, exclusive of those of the defective strips, are quite con-

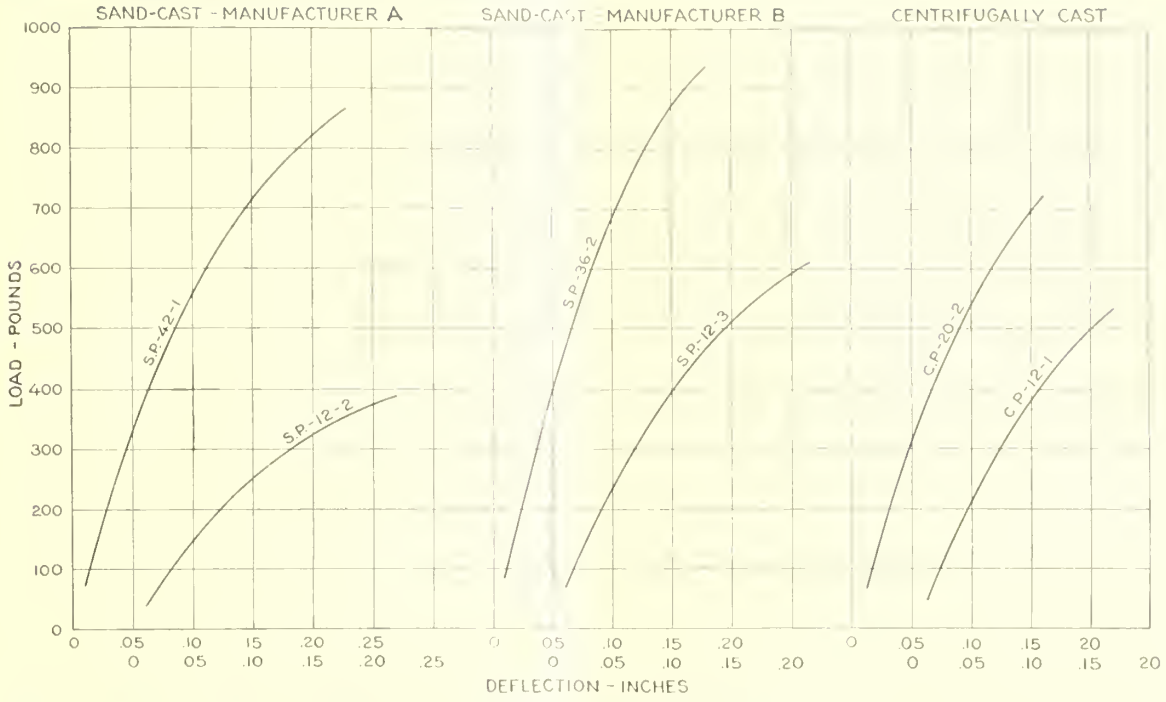

FIGURE 12.—TYPICAL LOAD-DEFLECTION CURVES OF STRIPS FROM SMOOTH CAST-IRON PIPE WITH PLAIN ENDS.

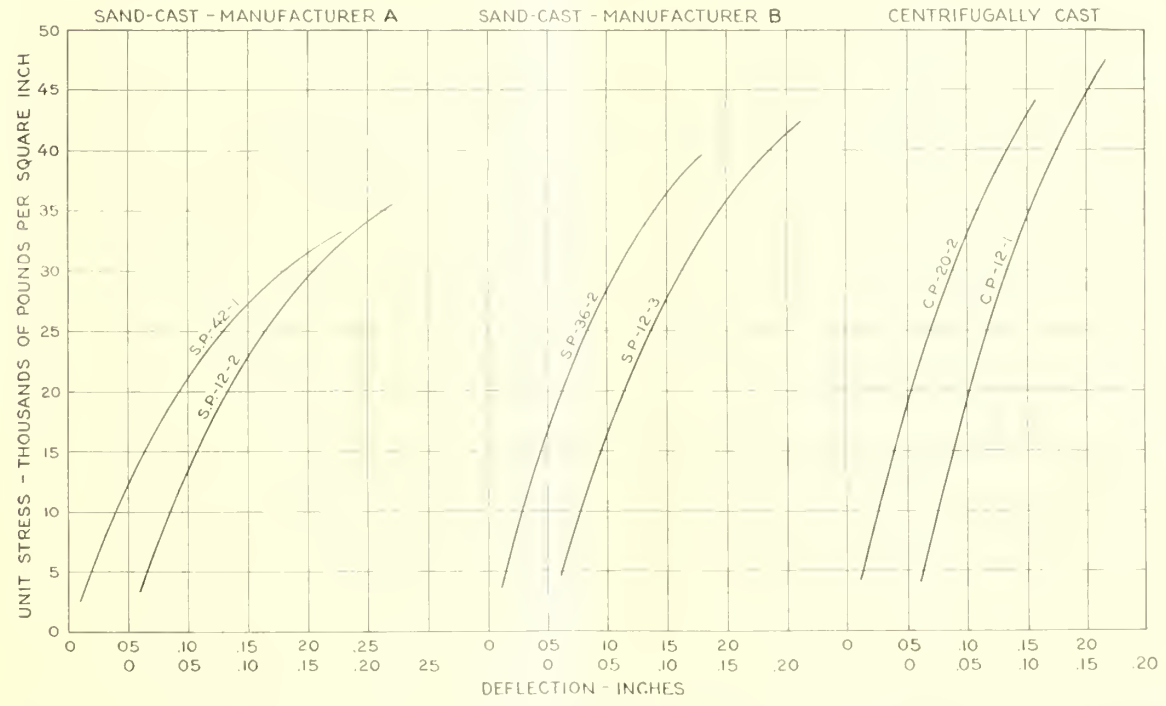

FIGURE 13.—TYPICAL STRESS-DEFLECTION CURVES OF STRIPS FROM SMOOTH CAST-IRON PIPE WITH PLAIN ENDS.

sistent and the average values show no significant differences between strips taken from areas of high stress and those taken from areas of low stress. Table 10 shows that none of the defective strips were taken near the crack nor at the bottom of the pipe, the two positions of greatest bending moment. Therefore, it was concluded that the test strips from the other pipe might safely be taken from the vicinity of the crack which had developed under the loading test, and this was done.

RESULTS OF STRIP TESTS DISCUSSED

The results obtained in the strip tests are given in tables 11, 12, and 13, and typical load-deflection and stress-deflection curves are shown in figures 12 and 13.

In general, 3 strips from each pipe specimen were tested but in a number of cases, where flaws were observed, additional specimens were machined and tested. In most cases flaws in the metal could not be detected by visual examination before the strips had been broken and in numerous cases the flaws

TABLE 10.—*Tests on strips from five positions in one 48-inch pipe*

[Modulus of rupture of pipe—31,800 pounds per square inch]

Location of strips in pipe	Modulus of elasticity		Modulus of rupture
	Origin	Failure	
	Million lb. per sq. in.	*Million lb. per sq. in.*	*Lb. per sq. in.*
Top	11.50	4.60	33,100
	10.35	4.65	32,500
	10.85	4.79	33,300
Average	10.90	4.68	33,000
45° from top	12.38	5.02	32,300
	11.85	4.89	31,900
	10.93	6.17	[1] 19,600
Average	12.12	4.96	32,100
90° from top	10.67	4.75	33,600
	8.64	5.97	[1] 22,400
	8.51	4.57	31,500
Average	9.59	4.66	32,600
45° from bottom	11.60	6.07	[1] 26,200
	11.20	4.77	31,900
	11.22	4.84	32,900
Average	11.21	4.80	32,400
Bottom	12.50	4.80	32,800
	12.46	4.93	33,200
	12.61	1.88	34,000
Average	12.52	4.87	33,300
Grand average	11.34	4.79	32,800

[1] Values for this strip excluded from averages.

which were observed had no apparent influence on the strength of the metal. For instance, 2 of the 3 strips from pipe SP-20-2 (table 11) had flaws and yet the modulus of rupture of both of these strips is higher than that of the strip in which no flaw was detected.

The test results for all strips, whether defective or not, are included in the tables and all results have been used in computing average values except in the case of the average values of the ratio of modulus of rupture of strips to modulus of rupture of pipe (column 11, table 11, columns 11 and 20, table 12, and column 11, table 13). The following explanation is given for this procedure which is at variance with that usually followed in experimental work of this nature.

The strip test has been proposed as a specification requirement to replace, at least to some extent, the more cumbersome and more expensive 3-edge bearing test on full-size specimens of pipe. If this is to be done, the strength developed by the strips should be indicative of the modulus of rupture of the pipe as determined in the 3-edge test. A study of the data shows that flaws in the strips in some cases resulted in a material reduction of the modulus of rupture and in other cases did not. The data also show that strips with flaws and of low strength were sometimes obtained from pipe of more than average strength and, conversely, that strips of more than average strength were sometimes obtained from pipe of relatively low strength. It appears that, in routine testing to determine compliance with specifications, low strength due to flaws in strips would not necessarily be indicative of low-strength pipe. It is felt that, in the study of the suitability of the strip test for the proposed purpose, the test values of all specimens are of equal importance in arriving at conclusions.

As has been stated, the surfaces of the strips were ground to remove the coating of pitch and any adhering sand before testing. In the case of the sand-cast pipe of manufacturer A and the centrifugally cast pipe this did not result in appreciable reduction in the breadth of the specimen. However, the surfaces of the sand-cast pipe of manufacturer B contained a considerable amount of embedded sand and the removal of this by grinding reduced the breadth of the specimens by varying amounts, frequently as much as several hundredths

TABLE 11.—*Results of bending tests on strips from smooth sand-cast pipe with plain ends, manufacturer A*

Identification no.	Modulus of elasticity of strips				Modulus of rupture of strips	Average	Deviation from grand average	Modulus of rupture of pipe	Ratio of modulus of rupture of strips to modulus of rupture of pipe		Remarks
	At origin	Average	At failure	Average							
1	2	3	4	5	6	7	8	9	10	11	
	Million lb. per sq. in.	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Per cent*	*Lb. per sq. in.*		*Average*	
SP-12-1	13.37	13.44	9.13	8.86	35,100	36,600	2.8	42,300	0.83	0.87	
	13.94		9.22		36,500		1.1		.86		
	13.01		8.24		38,300		6.1		.91		
2	11.06	11.83	8.44	7.29	26,400	32,200	26.9	40,300	[2] .66	[2] .87	Flaw at break.
	[3] 11.79		6.71		36,400		.8		.90		
	12.63		6.71		33,800		6.4		.84		
3	11.26	11.18	5.66	6.50	35,900	34,600	.6	39,700	.90	[2] .91	
	10.97		7.10		31,500		12.7		[2] .79		Do.
	11.30		6.75		36,300		.6		.91		
Average	12.15		7.55		34,500			40,800	.85	[2] .88	
SP-16-1	12.76	12.77	6.92	7.27	41,900	39,800	16.1	35,900	1.17	1.11	
	12.98		7.88		37,400		3.6		1.04		
	12.56		7.03		40,000		10.8		1.11		Broke 0.05 inch outside middle third.
2	11.88	12.77	7.15	7.27	38,600	40,200	6.9	39,100	.99	1.03	Broke 0.23 inch outside middle third.
	12.46		7.33		41,600		15.2		1.06		
	13.96		7.33		40,400		11.9		1.03		
3	9.57	10.00	6.20	6.10	35,800	37,700	.8	38,000	.94	.99	
	9.50		5.94		37,600		4.2		.99		
	10.94		6.15		39,800		10.2		1.05		Specimen warped.
Average	11.85		6.88		39,200			37,700	1.04	1.04	

TABLE 11.—*Results of bending tests on strips from smooth sand-cast pipe with plain ends, manufacturer A*—Continued

Identification no.	Modulus of elasticity of strips				Modulus of rupture of strips	Average	Deviation from grand average	Modulus of rupture of pipe	Ratio of modulus of rupture of strips to modulus of rupture of pipe		Remarks
	At origin	Average	At failure	Average							
1	2	3	4	5	6	7	8	9	10	11	
	Million lb. per sq. in.	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Lb. per sq. in*	*Lb. per sq. in.*	*Per cent*	*Lb. per sq. in.*		*Average*	
SP-20-1	11.59	11.98	5.67	6.07	40,200	38,800	11.4	43,300	0.93	0.90	
	12.39		6.22		37,200		3.0		.86		Flaw at break.
	11.95		6.31		38,900		7.8		.90		Specimen warped
2	12.75	13.04	6.21	6.37	40,400	39,200	11.9	43,700	.92	.90	Flaw at break.
	13.11		6.69		36,300		.6		.83		
	13.25		6.22		40,800		13.0		.93		Do.
3	14.11	13.81	6.61	6.64	39,100	37,800	8.3	37,300	1.05	1.01	
	13.36		6.52		37,200		3.0		1.00		
	13.95		6.80		37,100		2.8		.99		
Average	12.94		6.36		38,600			41,400	.93	.93	
SP-24-1	11.95	12.86	8.18	7.74	29,200	34,500	19.1	32,800	*.89	[1] 1.09	Flaw at break. Broke 0.23 inch outside middle third.
	12.37		7.05		35,400		1.9		1.08		Flaw at break.
	12.33		6.95		35,800		.8		1.09		Do.
	13.69		8.27		35,000		3.0		1.07		Do.
	13.17		8.45		34,800		3.6		1.06		Do.
	13.64		7.53		37,000		2.5		1.13		Do.
2	12.77	12.92	8.23	6.99	33,600	37,500	6.9	38,400	*.87	[1] 1.00	Specimen cracked.
	13.00		6.76		37,200		3.0		.97		Flaw at break.
	12.33		6.87		35,900		.6		.94		Do.
	12.92		6.83		37,900		5.0		.99		Do.
	13.81		6.29		43,800		21.3		1.14		
	12.69		6.96		36,700		1.7		.96		
3	10.82	11.48	6.23	6.42	38,300	38,100	6.1	43,400	.88	.88	
	12.50		6.40		38,700		7.2		.89		
	11.13		6.64		37,200		3.0		.86		
Average	12.61		7.18		36,400			38,200	.95	[1] .97	
SP-30-1	10.35	11.20	5.17	4.98	38,400	38,600	6.4	39,800	.96	.97	
	11.67		4.96		38,600		6.9		.97		
	11.58		4.82		38,800		7.5		.97		
2	10.59	10.28	4.61	4.57	33,900	34,200	6.1	32,000	1.06	1.07	
	10.00		4.46		34,000		5.8		1.06		
	10.24		4.63		34,600		4.2		1.08		
3	10.89	10.83	5.29	5.08	33,200	34,500	8.0	31,500	1.05	1.10	
	10.56		4.92		34,000		5.8		1.08		
	11.04		5.02		36,400		.8		1.16		
Average	10.77		4.88		35,800			34,400	1.04	1.04	
SP-36-1	11.33	11.58	6.00	6.06	37,700	38,100	4.4	35,600	1.06	1.07	
	11.56		6.20		38,800		7.5		1.09		
	11.84		5.98		37,900		5.0		1.06		
3	10.16	10.77	6.87	5.78	21,000	29,100	41.8	33,700	*.62	[1] .98	Specimen cracked.
	11.07		5.30		32,400		10.2		.96		
	11.08		5.17		33,900		6.1		1.01		
Average	11.17		5.92		33,600			34,600	.97	[1] 1.04	
SP-42-1	[2] 11.53	12.07	6.21	6.10	33,900	37,600	6.1	36,500	.93	1.03	
	12.19		6.13		38,700		7.2		1.06		
	12.49		5.97		40,300		11.6		1.10		
2	10.30	10.73	4.54	4.55	33,400	33,000	7.5	35,700	.94	.92	
	10.84		4.65		33,100		8.3		.93		
	11.06		4.45		32,500		10.0		.91		
3	12.80	12.79	5.41	5.32	37,100	37,100	2.8	40,800	.91	.91	
	12.80		5.32		37,100		2.8		.91		
	12.78		5.22		37,100		2.8		.91		
Average	11.87		5.32		35,900			37,700	.95	.95	
SP-48-1	11.50	10.90	4.60	4.68	33,100	33,000	8.3	31,800	1.04	1.04	
	10.35		4.65		32,500		10.0		1.02		
	10.85		4.79		33,300		7.8		1.05		
2	10.02	10.12	5.08	5.07	33,200	33,000	8.0	36,100	.92	.91	
	10.35		5.06		33,300		7.8		.92		
	10.00		5.08		32,500		10.0		.90		
3	11.80	11.69	5.69	5.63	35,900	36,200	.6	32,700	1.10	1.11	
	10.89		5.69		36,100		.0		1.10		
	12.38		5.51		36,700		1.7		1.12		
Average	10.90		5.13		34,100			33,500	1.02	1.02	
Grand average	11.87		6.24		36,100		7.0	37,400	.97	[1] .98	

[1] Values marked with asterisk (*) omitted from this average.
[2] Deflection data for this strip shown in figs. 12 and 13.

TABLE 12.—*Results of bending tests on strips from smooth sand-cast pipe, plain ends, manufacturer B*

Identification no.	Unground strips									
	Modulus of elasticity of strips				Modulus of rupture of strips	Average	Deviation from grand average	Modulus of rupture of pipe	Ratio of modulus of rupture of strips to modulus of rupture of pipe	
	At origin	Average	At failure	Average						
1	2	3	4	5	6	7	8	9	10	11
	Million lb. per sq. in.	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Lb. per sq. in.*		*Average*
SP-12-1	14.42	15.39	7.82	7.66	46,200	48,300	20.9	31,100	1.48	1.5
	15.93		7.57		49,200		28.8		1.58	
	15.82		7.60		49,600		29.8		1.59	
2	14.29	14.07	7.98	7.74	45,000	44,500	17.8	32,100	1.40	1.3
	14.68		7.41		45,400		18.8		1.41	
	13.24		7.84		43,000		12.6		1.34	
3	13.87	12.99	7.88	8.41	41,800	38,200	9.4	37,300	1.12	[1] 1.0
	13.65		8.08		40,700		6.5		1.09	
	13.08		8.65		38,100		.3		1.02	
	12.54		7.83		41,900		9.7		1.12	
	12.73		9.27		33,600		12.0		*.90	
	12.67		7.60		38,600		1.0		1.03	
	12.41		9.54		32,500		14.9		*.87	
Average	13.79		8.08		42,000			33,500	1.25	[1] 1.3
SP-18-1	15.30	15.40	8.92	9.44	46,900	45,500	22.8	27,300	1.72	[1] 1.7
	15.32		8.87		48,700		27.5		1.78	
	15.58		10.54		40,800		6.8		*1.49	
2	13.38	13.72	8.71	8.77	34,200	38,300	10.5	36,900	.93	1.0
	14.25		8.98		41,100		7.6		1.11	
	13.53		8.62		39,700		3.9		1.08	
3	13.82	13.91	9.33	8.84	40,600	39,900	6.3	40,500	1.00	.9
	14.38		8.78		39,900		4.5		.99	
	13.52		8.42		39,100		2.4		.97	
Average	14.34		9.02		41,200			34,900	1.18	[1] 1.1
SP-24-1	14.90	14.62	8.43	8.35	41,500	39,700	8.6	31,500	1.32	1.2
	14.05		8.08		38,600		1.0		1.23	
	14.92		8.54		39,100		2.4		1.24	
2	14.18	14.37	7.40	7.76	43,600	42,400	14.1	37,200	1.17	1.1
	14.26		7.73		41,700		9.2		1.12	
	14.67		8.15		42,000		9.9		1.13	
3	12.92	13.28	8.64	9.36	35,300	31,100	7.6	37,500	.94	[1] .9
	13.18		8.47		32,700		14.4		.87	
	13.76		9.75		27,300		28.5		*.73	
	13.38		9.52		32,700		14.4		.87	
	13.09		10.52		24,000		37.2		*.64	
	13.32		9.27		34,500		9.7		.92	
Average	13.89		8.71		36,100			35,400	1.02	[1] 1.0
SP-30-1	12.88	14.02	8.52	9.08	34,300	32,600	10.2	33,800	1.01	[1] 1.0
	14.68		8.64		34,500		9.7		1.02	
	14.49		10.09		29,000		24.1		*.86	
2	12.96	12.96	8.12	7.95	32,700	35,100	14.4	35,000	.93	1.0
	12.10		7.40		36,000		5.8		1.03	
	13.82		8.33		36,500		4.5		1.04	
3	13.96	13.45	7.50	8.23	37,200	33,400	2.6	32,100	1.16	1.0
	12.21		8.57		30,100		21.2		*.94	
	14.19		8.62		32,800		14.1		1.02	
Average	13.48		8.42		33,700			33,600	1.00	[1] 1.0
SP-36-1	13.20	13.37	6.80	6.89	35,700	34,700	6.5	34,500	1.03	1.0
	13.35		7.20		33,900		11.3		.98	
	13.55		6.66		34,600		9.4		1.00	
2	13.85	14.05	8.59	7.98	36,900	38,700	3.4	33,800	1.09	1.1
	13.76		7.34		40,100		5.0		1.19	
	14.53		8.00		39,000		2.1		1.15	
Average	13.71		7.43		36,700			34,200	1.07	1.0
Grand average	13.85		8.39		38,200		11.8	34,300	1.11	[1] 1.1

[1] Values marked with asterisk (*) are omitted from this average.

of an inch. Moduli of rupture computed on the basis of the dimensions of the ground strips are undoubtedly more indicative of the strength of the metal itself than are similar values computed on the basis of the dimensions of the strips before grinding. However, moduli computed on the latter basis are more directly comparable to the moduli reported for the pipe specimens, since these were calculated on the basis of the gross thickness of shell, including embedded sand. Therefore, in table 12 which shows the results obtained on strips from the pipe of manufacturer B, values of modul of elasticity and rupture are reported for both ungrounc and ground strips. The tests were made on the strips after grinding, and no tests were actually made or unground strips.

The moduli of elasticity and rupture reported fo unground strips were obtained by using the loads and deflections of the ground strips and the dimensions o the strips in their original condition including the thick ness of embedded sand. Since the grinding operatio

TABLE 12.—*Results of bending tests on strips from smooth sand-cast pipe, plain ends, manufacturer B* Continued

Identification no.	Ground strips									Remarks
	Modulus of elasticity of strips				Modulus of rupture of strips	Average	Deviation from grand average	Ratio of modulus of rupture of strips to modulus of rupture of pipe		
	At origin	Average	At failure	Average						
1	12	13	14	15	16	17	18	19	20	
	Million lb. per sq. in.	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*		*Average*	
SP-12-1	14.70	15.68	7.97	7.81	47,100	49,300	16.0	1.51	1.59	
	16.23		7.71		50,100		23.4	1.61		
	16.11		7.74		50,600		24.6	1.63		
2	14.57	14.35	8.14	7.90	45,900	45,400	13.1	1.43	1.41	
	14.98		7.56		46,400		14.3	1.45		
	13.51		8.00		43,900		8.1	1.37		
3	14.37	13.51	8.16	8.74	43,300	39,700	6.7	1.16	[1] 1.12	Flaw at break.
	[2] 14.14		8.37		42,200		3.9	1.13		Do.
	13.54		8.95		39,400		3.0	1.06		Do.
	13.08		8.17		43,700		7.6	1.17		Do.
	13.29		9.68		35,100		13.5	*.94		Do.
	13.20		7.92		40,200		1.0	1.08		Flaw at break. Broke 0.4 inch outside middle third.
	12.94		9.95		33,900		16.5	*.91		Flaw at break.
Average	14.20		8.33		43,200			1.29	[1] 1.34	
SP-18-1	15.98	16.08	9.31	9.86	49,000	47,500	20.7	1.79	[1] 1.83	
	15.99		9.26		50,900		25.4	1.86		
	16.27		11.00		42,600		4.9	* 1.56		Do.
2	14.09	14.44	9.17	9.23	35,900	40,300	11.6	.97	1.09	
	15.00		9.45		43,200		6.4	1.17		Broke 0.14 inch outside middle third.
	14.24		9.07		41,800		3.0	1.13		
3	15.50	15.58	10.46	9.91	45,600	44,700	12.3	1.13	1.10	
	16.15		9.87		44,700		10.1	1.10		Broke 0.42 inch outside middle third.
	15.08		9.39		43,700		7.6	1.08		
Average	15.37		9.66		44,200			1.27	[1] 1.27	
SP-24-1	15.96	15.67	9.03	8.95	44,500	42,600	9.6	1.41	1.35	
	15.06		8.66		41,300		1.7	1.31		Broke 0.23 inch outside middle third.
	15.98		9.15		41,900		3.2	1.33		
2	15.11	15.31	7.89	8.27	46,500	45,200	14.5	1.25	1.22	Specimen warped.
	15.19		8.23		44,400		9.4	1.19		
	15.64		8.69		44,700		10.1	1.20		
3	13.80	14.17	9.23	10.00	37,700	33,200	7.1	1.01	1.96	Flaw at break.
	14.07		9.04		35,000		13.8	.93		Do.
	14.69		10.41		29,100		28.3	*.78		Do.
	14.29		10.17		34,900		14.0	.93		Flaw at break. Broke 0.15 inch outside middle third.
	13.97		11.23		25,600		36.9	*.68		
	14.22		9.90		36,800		9.4	.98		Flaw at break. Broke 0.05 inch outside middle third.
Average	14.83		9.30		38,500			1.09	[1] 1.15	
SP-30-1	13.26	14.43	8.77	9.35	35,400	33,600	12.8	1.05	[1] 1.05	
	15.12		8.90		35,500		12.6	1.05		
	14.92		10.39		29,900		26.4	*.88		Flaw at break.
2	14.40	14.39	9.02	8.83	36,400	39,000	10.3	1.04	1.11	Do.
	13.45		8.22		40,000		1.5	1.14		
	15.33		9.24		40,500		.2	1.16		
3	15.48	14.92	8.32	9.13	41,200	37,000	1.5	1.28	[1] 1.21	
	13.55		9.51		33,400		17.7	* 1.04		Do.
	15.74		9.56		36,400		10.3	1.13		
Average	14.58		9.10		36,500			1.09	[1] 1.13	
SP-36-1	14.88	15.08	7.67	7.77	40,200	39,100	1.0	1.17	1.13	
	15.06		8.12		38,200		5.9	1.11		
	15.30		7.51		39,000		3.9	1.13		Broke 0.13 inch outside middle third.
2	[2] 14.77	14.97	9.16	8.50	39,400	41,300	3.0	1.17	1.22	
	14.66		7.82		42,800		5.4	1.27		
	15.49		8.52		41,600		2.5	1.23		
Average	15.03		8.13		40,200			1.18	1.18	
Grand average	14.74		8.93		40,600		10.8	1.18	[1] 1.22	

[2] Deflection data for this strip shown in Figures 12 and 13.

unavoidably removed small amounts of surface metal, the strips in their original or unground condition probably would have supported somewhat greater loads than they did in the tests which were made. Therefore, the moduli reported for the unground strips in the first part of table 12 are probably somewhat lower than the true values but it is not believed that the error involved is large enough to be of any importance.

All average values given in tables 11, 12, and 13 are weighted averages and are based on all the test results, irrespective of defects in individual specimens, with the exception of the average values given in column 11 of table 11, columns 11 and 20 of table 12 and column 11 of table 13. In these columns are given the averages of the ratios of modulus of rupture of strips to modulus of rupture of pipe, and the moduli of strips manifestly

TABLE 13.—*Results of bending tests on strips from smooth centrifugally cast pipe with plain ends*

Identification no.	Modulus of elasticity of strips				Modulus of rupture of strips	Average	Deviation from grand average	Modulus of rupture of pipe	Ratio of modulus of rupture of strips to modulus of rupture of pipe[1]		Remarks
	At origin	Average	At failure	Average							
1	2	3	4	5	6	7	8	9	10	11	
	Million lb. per sq. in.	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Million lb. per sq. in.*	*Lb. per sq. in.*	*Lb. per sq. in.*	*Percent*	*Lb. per sq. in.*		*Average*	
CP-12-1	16.22	16.32	13.15	12.38	52,000	52,400	13.5	50,000	1.04	1.05	
	[2] 16.17		11.71		51,100		11.6		1.02		
	16.57		12.27		54,000		17.9		1.08		
2	16.99	17.47	14.03	13.25	41,400	44,100	9.6	54,700	.76	.81	
	17.49		13.54		45,400		.9		.83		
	17.94		12.18		45,500		.7		.83		
3	14.40	14.51	11.81	12.40	52,200	44,500	14.0	53,600	.97	1.93	
	14.16		12.55		47,100		2.8		.88		Specimen warped.
	14.98		12.84		34,100		25.6		*.64		Specimen warped. Broke 0.9 inch outside middle third.
Average	16.10		12.68		47,000			52,800	.89	1.92	
CP-16-1	15.09	15.39	12.07	12.63	44,100	42,300	3.7	55,700	.79	.76	
	14.43		11.73		41,800		8.7		.75		Broke 0.07 inch outside middle third.
	16.66		14.10		40,900		10.7		.73		
2	16.75	16.87	11.12	10.85	51,600	50,500	12.7	59,700	.86	.85	
	16.70		11.27		48,400		5.7		.81		
	17.16		10.17		51,500		12.4		.86		
3	16.35	16.77	11.48	11.43	45,400	45,700	.9	51,500	.88	.89	
	16.34		11.21		46,200		.9		.90		
	17.62		11.61		45,400		.9		.88		
Average	16.34		11.64		46,100			55,600	.83	.83	
CP-20-1	16.87	16.54	9.49	10.15	53,800	45,200	17.5	44,700	1.20	1.18	
	16.09		10.56		30,400		33.6		*.68		Specimen cracked
	16.66		10.41		51,300		12.0		1.15		
2	15.58	15.70	11.74	11.47	45,500	46,200	.7	49,000	.93	.94	
	15.51		10.84		48,100		5.0		.98		
	[2] 16.00		11.83		45,100		1.5		.92		
3	15.94	16.08	11.45	11.86	43,200	41,900	5.7	52,100	.83	.80	
	15.70		11.83		41,800		8.7		.80		
	16.61		12.31		40,600		11.4		.78		
Average	16.11		11.16		44,400			48,600	.91	1.95	
Grand average	16.18		11.83		45,800		9.2	52,300	.88	1.90	

[1] Values marked with asterisk (*) are omitted from the average.

[2] Deflection data for this strip shown in figures 12 and 13.

defective have been omitted in order to show the small effect on the grand averages of these columns as compared with the grand averages of the columns immediately preceding which are based on all test results. The omission of test results in the computation of these averages was based entirely on personal judgment. If the modulus of a strip was materially lower than the average of the other strips from that pipe, it was discarded; otherwise it was retained.

For each strip, two values of modulus of elasticity are reported; the secant modulus at failure and the tangent modulus at the origin. Of the two, the secant modulus is more suitable for use as a specification requirement on account of the greater ease and accuracy of its determination. The modulus at the origin is of some interest in showing the maximum value of modulus of elasticity which may be obtained in this test.

The average modulus of rupture of strips taken from the centrifugally-cast pipe is relatively high as was the case in the tests on full-size specimens. It may also be noted that in the tests of pipe the sand-cast pipe of manufacturer A developed a higher average modulus than the sand cast pipe of manufacturer B, while in the strip tests the reverse is true. The average modulus of 45,800 pounds per square inch for the strips of centrifugally cast metal may be compared with an average modulus of 38,200 pounds per square inch for the unground strips of sand-cast metal of manufacturer B and 36,100 pounds per square inch for the strips from the sand-cast metal of manufacturer A. The ground strips from the sand-cast pipe of manufacturer B developed an average modulus of rupture of 40,600 pounds per square inch. The inclusion, in the computations, of the thickness of the embedded surface sand had the effect of reducing the average modulus about 6 percent to 38,200 pounds per square inch.

The consistency of the values of modulus of rupture developed by individual strips is indicated by the average deviations of 7.0 percent, 9.2 percent, and 11.8 percent, respectively, for the sand-cast metal of manufacturer A, the centrifugally cast metal and the unground strips of the sand-cast metal of manufacturer B. The relatively low average deviation of the moduli of rupture developed in the tests of centrifugally cast pipe may be thought to be indicative of a more uniform product, but this indication is not corroborated by the above results from tests on the metals themselves. The moduli of rupture of the ground strips (table 12) are somewhat more consistent than those of the unground strips. This is to be expected since the low-strength material ground from the surface of the strips was not of uniform thickness.

STRIP TESTS NOT A SATISFACTORY INDICATION OF PIPE STRENGTH

As in the case of the pipe specimens, the moduli of rupture of individual strips vary through a rather wide range. For the sand-cast strips of manufacturer A the modulus of rupture varies from a minimum of 21,000

pounds per square inch to a maximum of 43,800 pounds per square inch with a maximum deviation from the average of 41.8 percent. If the defective specimens are excluded, the range is from 32,400 to 43,800 pounds per square inch. For the unground sand-cast strips of manufacturer B, the variation is from 24,000 to 49,600 with a maximum deviation of 37.2 percent. Exclusive of defective specimens the range is from 32,700 to 49,600 pounds per square inch. Strips from the centrifugally cast pipe give a range in values of moduli of rupture from 30,400 to 54,000 pounds per square inch with a maximum deviation of 33.6 percent. Exclusive of specimens manifestly defective the range is from 40,600 to 54,000 pounds per square inch.

In the strip tests there is the same general, though far from consistent trend toward somewhat lower values of moduli of rupture with increasing thickness of metal as was the case in the pipe tests. However, the relationship is too vague to be of practical significance.

Both the tangent modulus of elasticity at origin and the secant modulus at failure, show the centrifugally cast metal to be considerably stiffer than the sand-cast metal. Average values for the tangent modulus at origin are 11,870,000 pounds per square inch, 13,850,000 pounds per square inch and 16,180,000 pounds per square inch, respectively, for the sand-cast metals of manufacturer A, manufacturer B, and the centrifugally cast metal. Corresponding values for the secant modulus at failure are 6,240,000, 8,390,000, and 11,830,000 pounds per square inch. Here, also, there is some indication of a general trend toward decreasing values with increasing thickness of metal, which appears to be somewhat more pronounced for the modulus at failure than for the modulus at origin, and more pronounced in the metal of manufacturer A and the centrifugally cast metal than in the metal of manufacturer B.

Possibly the most important data resulting from the strip tests are the calculated ratios of the modulus of rupture of each strip to the modulus of rupture of the pipe from which it was taken, and the average values of these ratios.

Including all strips, the values of the modulus ratio for the sand-cast pipe of manufacturer A range from 0.62 to 1.17, with an average of 0.97. The exclusion of badly defective strips changes the range to 0.83 to 1.17 and the average to 0.98. The ratios for the centrifugally cast pipe range from 0.64 to 1.20, with an average of 0.88 for all strips and, exclusive of defective specimens, an average of 0.90 with a range of 0.73 to 1.20.

The modulus ratios for the unground strips from the sand-cast pipe of manufacturer B range from 0.64 to 1.78, with an average of 1.11, and the range for the ground strips is from 0.68 to 1.86, with an average of 1.18. The exclusion of defective strips changes the ranges of individual values to 0.87 to 1.78 and 0.93 to 1.86, respectively, with average values of 1.15 and 1.22.

It will be observed that there is a considerable difference between the average values of the modulus ratios for the three groups of pipe. It will also be observed that the exclusion of the ratios for defective strips does not alter the average values materially and that there is the same general relationship between them irrespective of whether the defective strips are included or discarded.

It is, perhaps, to be expected that the average modulus ratio for the centrifugally cast pipe should differ from those for the sand-cast pipe, since the physical characteristics of the metals themselves differ considerably. However, if the strength of strips is to be accepted as a measure of the strength of the pipe from which they are taken, a reasonably close agreement should exist between the modulus ratios for two sets of pipe cast in a similar manner and of metal of similar characteristics. This agreement does not exist for the sand-cast pipe, the average modulus ratios for all specimens being 0.97 for manufacturer A and 1.11 for the unground strips of manufacturer B.

It has been pointed out that (1) obviously defective strips may be obtained from pipe of more than average strength; (2) that strips of relatively high strength may be obtained from pipe of less than average strength; (3) that there is a wide range in the individual values of the modulus ratios; and (4) that there is lack of agreement between the average modulus ratios for two groups of pipe of the same general character. These facts lead to the conclusion that the results of tests on small strips cut from pipe are not acceptable indices of the strength which the pipe may be expected to develop in the 3-edge bearing test.

USE OF TEST RESULTS IN PREPARATION OF SPECIFICATIONS

During the progress of the investigation which has been described, the results as they were obtained were made available from time to time to a committee of the American Society for Testing Materials which was engaged in the preparation of tentative specifications for cast-iron culvert pipe. These specifications, A.S.T.M. Designation: A142-32T, which have now been published as tentative standards of the society, are believed to be the first specifications for cast-iron culvert pipe in which the required shell thicknesses have been established on a rational basis.

These tentative specifications recognize three classes of pipe known as standard, heavy, and extra-heavy pipe for which the required minimum strengths in the 3-edge bearing test are, respectively, 2,000 D, 3,000 D, and 4,000 D pounds per foot of laying length.

The shell thicknesses for smooth pipe were calculated on the basis of equation 9 for a modulus of rupture of 30,000 pounds per square inch and the thicknesses so determined were arbitrarily increased by 10 percent. Table 14 gives the specified shell thicknesses and their derivation.

TABLE 14.—*Shell thickness of smooth cast-iron culvert pipe as required by A.S.T.M. tentative standard specification, A142-32T*

Nominal diameter of pipe (inches)	Standard pipe (2,000 D)			Heavy pipe (3,000 D)			Extra-heavy pipe (4,000 D)		
	Theoretical thickness [1]	Theoretical thickness plus 10 percent	Specified thickness	Theoretical thickness [1]	Theoretical thickness plus 10 percent	Specified thickness	Theoretical thickness [1]	Theoretical thickness plus 10 percent	Specified thickness
	Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
12	0.255	0.281	0.37	0.313	0.344	0.37	0.362	0.398	0.40
14	.298	.328	.37	.365	.402	.40	.423	.465	.46
16	.340	.374	.40	.417	.459	.46	.483	.531	.53
18	.383	.421	.42	.470	.517	.52	.543	.597	.60
20	.425	.468	.47	.522	.574	.57	.604	.664	.66
24	.510	.561	.56	.626	.689	.69	.724	.796	.80
30	.638	.702	.70	.783	.861	.86	.905	.996	1.00
36	.765	.842	.84	.939	1.033	1.03	1.086	1.195	1.20
42	.893	.982	.98	1.096	1.206	1.20	1.268	1.395	1.40
48	1.020	1.122	1.12	1.252	1.378	1.38	1.449	1.594	1.60

[1] Theoretical thicknesses calculated for modulus of rupture of 30,000 pounds per square inch.

It will be noted that the specified thicknesses of the 12-, 14-, and 16-inch standard pipe and of the 12-inch heavy pipe are somewhat greater than the theoretical thicknesses increased by 10 percent. In these cases arbitrary increases were made in order that the required thickness of metal be not less than three eighths of an inch.

If the moduli of rupture obtained in the tests which have been reported may be considered generally representative of the values which would be obtained with any commercially available cast-iron culvert pipe, then the modulus of 30,000 pounds per square inch, which is the basis of design in the tentative specifications, may be considered somewhat conservative since it is somewhat lower than the average values which were obtained with sand-cast pipe. However, the modulus of one pipe tested was less than 30,000 pounds per square inch and the moduli of a considerable proportion were between 30,000 and 35,000 pounds per square inch. In any case, any conservatism in the matter of unit stress is balanced to some extent by the provision of the specification which permits a maximum underrun in thickness of 15 percent.

CONCLUSIONS

The following conclusions, based on the results obtained in this investigation, are presented:

1. In the 3-edge bearing test for pipe with bell ends it is preferable to locate the point or center of load application, on the upper bearing block, at the center of pipe rather than at the center of length of bearing.

2. The design of cast-iron pipe to withstand the 3-edge bearing test may more safely be based on the results of tests on specimens with plain ends than on the results obtained on specimens with bell ends, due to the variable and uncertain influence of the bell ends on the supporting strength.

3. The pipe included in this investigation which were centrifugally cast in metal contact molds were of considerably higher strength than the sand-cast pipe.

4. Spiral corrugated pipe and ribbed pipe, similar in all respects to those included in this investigation, may be expected to develop a strength in the 3-edge bearing test of at least 2,000 D pounds per linear foot of laying length.

5. The magnitude of the vertical deflections of pipe under a given load in the 3-edge bearing test is influenced by the type of design, the quality of the metal, and the details of design. For the smooth pipe included in this study the magnitude of these deflections was influenced more by variations in the ratio of shell thickness to diameter than by variations in the modulus of elasticity of the metal.

6. The modulus of rupture of small strips cut from a specimen of pipe is not a good index of the modulus of rupture which the pipe will develop in the 3-edge bearing test.

RECOMMENDATIONS

In addition to the above conclusions, it is desired to offer the following recommendations with respect to the design of smooth cast-iron pipe:

1. That the required minimum shell thickness be determined on a rational basis such as is provided by the theoretical equation (9) which has been presented.

2. That the selection of the modulus of rupture to be used in design be made on the basis of the modulus which may be expected in pipe manufactured to meet the design requirements.

3. That, as a safeguard against unavoidable inaccuracies in manufacture, the required nominal shell thickness be made greater than the theoretical thickness as determined by equation (9), by an arbitrary allowance of not less than 10 percent.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS I—PROJECTS ON THE FEDERAL-AID HIGHWAY SYSTEM OUTSIDE OF MUNICIPALITIES

AS OF OCTOBER 31, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	4.185.067					1.019.052.26	560.478.75	458.573.51	3.9	38.1	1.475.770.69	154.7	2.148.817.56
Arizona	3.804.731					1.736.625.07	1.734.470.83		7.8	109.6	787.782.12	59.9	1,282,475.05
Arkansas	3.374.167					521.166.78	268.090.09	253.076.69	1.1	3.5	642.812.51	47.0	2.463.264.40
California	7.803.677					4.695.867.16	4.020.372.77		2.2	136.0	1.596.955.68	77.1	2.186.348.55
Colorado	3,437.265					1.019.726.64	1.019.726.64		29.9	54.1	586.461.09	23.1	1.831.077.27
Connecticut	1.404.213					202.625.72	202.625.72		1.5	2.9	60.380.55	.1	1.141.206.73
Delaware	909.544					399.126.30	399.126.30		35.3	19.7	346.978.00	7.5	163,439.70
Florida	2.615.917					1.381.054.43	759.579.92	621,474.51	11.6	50.8	737.828.54	32.9	1.118,508.54
Georgia	5.045.592										582.663.17	23.0	4.462.928.53
Idaho	2.243.125					1.491.706.95	1,488,098.73		18.0	116.5	105.229.71	13.7	649.796.56
Illinois	4.431.348										229.859.90	5.9	4,201.488.10
Indiana	4.717.786					1.174.800.00	1,174,800.00		6.6	46.4	1.219.269.59	34.9	2,323,716.41
Iowa	5.027.830					3.755.606.76	3.533,500.00		29.8	160.2	1,315.400.00	78.5	178.930.00
Kansas	5.044.802					371.785.29	371.785.29		6.2	48.7	2.498.858.55	269.5	2.174.158.16
Kentucky	3.608.332					1.121.901.26	1.121.901.26		.4	82.9	961.119.80	73.4	1,525.310.94
Louisiana	2.914.295					488.288.02	488.288.02		16.8	18.8	1.213.617.51	24.5	1,212.389.47
Maine	1,684.959					802.228.54	802.228.54		27.5	27.8	257.546.33	4.6	625.184.13
Maryland	1.782.263					156.305.93	146.657.18		7.5	4.8	674.613.95	10.0	960.991.87
Massachusetts	1.932.950					380.636.86	195.937.51	179.984.24	43.9	13.1	418.450.25	17.7	1,318.562.24
Michigan	5.094.491					1.069.450.00	1.069,450.00		34.6	69.5	521,700.00	42.3	3.503.341.00
Minnesota	5.115.153					2.069.289.35	2.069.289.33		37.7	359.3	168.063.88	35.4	2.877.799.79
Mississippi	3.489.337					368.235.79	219.800.63	148.435.16	4.1	18.5	217.817.13	25.0	3,051.719.24
Missouri	6.090.153					1.650.581.70	1.519.778.85		5.4	56.7	1.804.212.95	74.9	2.766.161.20
Montana	3.719.874					3.486.487.61	3.386.487.61	100.000.00	9.4	259.3	137.493.00	13.4	275.893.39
Nebraska	3,914,481	43.132.00	39,210.94		18.2	2.712.212.78	1.988.005.06	108,538.00	30.9	234.1	819.843.36	66.3	1.067.421.64
Nevada	2,909.387					527.623.97	527.623.97		19.3	81.6	621.316.17	79.7	1.760.446.86
New Hampshire	954.919					439.426.36	406.183.50		15.5	7.9			548.735.50
New Jersey	3.065.137					327.297.82	327.297.82		8.6	6.1	28.948.83	.5	2.708.890.35
New Mexico	2.896.467					1.776.445.05	1,776,445.05		17.4	174.2	718,644.85	62.7	401.377.10
New York	10.830.099					8,130,519.00	7,827,060.00	300,000.00	8.2	174.9	1,993,650.00	23.1	1.009.389.00
North Carolina	4.761.147					803.595.81	401.797.95	401.797.86	29.9	94.8	1.123.833.23	158.3	3.235.515.82
North Dakota	2.902.224					872.509.98	872.509.98		31.6	359.3	1,243.530.25	343.8	786.183.77
Ohio	6.968.066					3.324.120.00	3.324.120.00		26.1	88.1	3,015,590.00	62.6	628.356.00
Oklahoma	4.608.399					1.775.719.78	1.775.719.78		3.6	114.4	1.807.422.99	99.8	1.025.256.23
Oregon	3.053.448					1.789.001.67	1.740.702.17		19.0	106.6	1.024.610.70	61.9	288.135.13
Pennsylvania	5.757.978					533.810.98	533.810.98			13.2	2.879.699.67	78.4	2.344.467.35
Rhode Island	999.354					952.043.50	952.043.50		14.7	19.8	42.490.08	.7	4.820.42
South Carolina	2.729.583					715.738.71	713.009.09	2.729.62	5.3	89.8	519.210.42	59.4	1.497.363.49
South Dakota	3.005.739					1.112.298.15	1.077.368.55	34,929.60	37.3	181.5	868.156.17	104.7	1.060.214.28
Tennessee	4,246,309					776.289.69	485.864.15	290.425.54	12.6	14.3	719.905.69	42.9	3.040.539.16
Texas	12.122.012					2.959.818.68	2.253.616.65		9.0	286.3	2.691,190.09	330.0	7.177.205.26
Utah	2.097.354					1.448.385.26	1.448.385.26		53.7	163.6	172.799.35	3.1	476.169.39
Vermont	931.919					611.678.48	611.678.48		13.4	29.0	91.944.79	9.2	228.295.73
Virginia	3.708.379					598.754.92	581.572.05	17.182.87	7.6	25.8	1,252.749.25	50.3	1.874.057.70
Washington	3.057.934					1.153.446.30	1.150.089.48	3.356.82	24.4	50.9	682.589.85	25.3	1.225.254.67
West Virginia	2,013.405					1,321.306.32	1.321.306.32		22.0	58.6	459.853.05	13.7	232.245.63
Wisconsin	4.862.441					2,564,447.01	2.542.947.01	21.500.00	34.1	109.1	422.745.42	29.3	1.896.748.57
Wyoming	2,250,663					1,773.022.95	1.629.300.00	60,000.00	21.4	355.1	324.885.99	76.0	296,477.01
District of Columbia													
Hawaii	1.683.956					87.001.76	43.500.88	43.500.88		5.1	157.010.70	5.1	1.483,444.42
TOTALS	185.807.671	43,132.00	39,210.94		18.2	68.449.063.35	62.784.431.65	3.045.505.30	17.7	4.541.3	42.243.505.80	2.935.8	80,740.522.61

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS II—PROJECTS ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES

AS OF OCTOBER 31, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	2,092,533										119,619.43	6.2	1,972,913.57
Arizona	781,794					11,976.30	11,976.30		8.4	1.0			769,817.70
Arkansas	1,687,084					17,350.27	9,963.18	7,387.09	10.0	.1	207,534.15	7.0	1,469,586.67
California	3,901,839					125,384.62	112,319.65		3.6	1.9	274,909.28	3.4	3,514,610.07
Colorado	1,718,633					105,469.21	105,469.21		62.6	3.0	132,295.57	3.1	1,480,868.22
Connecticut	802,407					369,505.30	369,505.30		1.9	3.2	237,835.98	2.0	195,065.72
Delaware	454,772					185,821.40	185,821.40		14.0	2.0	13,542.50		255,408.10
Florida	1,307,959					139,270.42	76,598.73	62,671.69	1.3	2.9	150,901.19	2.8	1,080,459.08
Georgia	2,724,620										12,325.93	.5	2,712,294.07
Idaho	1,121,562					39,947.35	39,947.35		7.5	3.0	14,616.01	.9	1,066,998.64
Illinois	6,877,199												6,877,199.00
Indiana	4,818,165										24,444.60	.4	4,793,720.40
Iowa	2,815,585					466,173.76	438,350.00		37.0	12.6	477,500.00	14.3	1,899,735.00
Kansas	2,522,401					23,085.32	23,085.32		8.7	.4	73,830.32	6.8	2,425,485.36
Kentucky	2,029,687										31,123.23	1.5	1,998,563.77
Louisiana	1,457,148					264,717.31	264,717.31		16.2	3.7	141,679.31	7.4	1,050,751.38
Maine	842,479					410,082.40	410,082.40		21.2	8.7	50,292.64	2.1	382,103.96
Maryland	891,132					16,788.76	12,638.76		7.9	.6			878,493.24
Massachusetts	4,136,382					385,758.23	339,511.37	40,000.00	15.6	5.1	1,202,687.05	3.8	2,594,183.58
Michigan	4,457,679					273,520.00	273,520.00		59.2	5.9	179,150.00	2.5	4,005,009.00
Minnesota	3,410,102					972,113.85	972,113.85		35.9	48.3	457,127.95	10.7	1,980,860.20
Mississippi	1,744,669					27,687.80	15,228.30	12,459.50		1.8	63,040.23	2.2	1,666,400.47
Missouri	3,045,077					532,146.89	524,278.41		7.4	4.0	253,602.45	4.8	2,267,196.14
Montana	1,859,937					5,275.14	5,275.14		37.9	.3	337,737.24	10.8	1,516,924.62
Nebraska	1,957,240	3,656.08	3,656.08		1.3	448,863.57	448,160.17		16.5	11.5	120,398.50	4.8	1,385,025.25
Nevada	500,051					50,304.27	50,304.27		77.5	1.2			449,746.73
New Hampshire	477,460					26,790.69	20,000.00		20.0	.3	66,313.99	1.5	391,146.01
New Jersey	3,217,442					671,930.61	671,930.61		16.8	5.4	1,349,899.25	7.7	1,195,612.14
New Mexico	1,448,234										234,312.35	5.4	1,213,921.65
New York	7,837,865					2,722,830.00	2,721,530.00		6.1	25.5	2,936,850.00	16.2	2,179,485.00
North Carolina	2,380,573					81,177.36	81,177.36		20.9	4.5	34,177.12	6.9	2,265,218.52
North Dakota	1,451,112					15,572.15	15,572.15		83.5	4.8	146,818.39	5.9	1,288,721.46
Ohio	4,645,378					806,110.00	716,135.00		8.1	9.0	463,400.00	6.6	3,465,843.00
Oklahoma	2,304,200					25,131.78	25,131.78		67.6	1.7	196,768.60	4.9	2,082,299.62
Oregon	1,526,724					190,977.53	190,977.53			4.0	90,442.00	2.1	1,245,304.47
Pennsylvania	5,416,051					130,791.78	130,791.78			3.5	1,053,237.00	21.6	4,232,022.22
Rhode Island	499,677												499,677.00
South Carolina	1,364,791					29,263.72	28,858.71	405.01		3.7	63,717.66	6.7	1,272,214.63
South Dakota	1,502,870					119,000.23	119,000.23		16.8	7.3	140,903.35	7.0	1,242,966.42
Tennessee	2,123,155					39,051.81	39,051.81		7.7	1.6	8,045.84	.5	2,076,057.35
Texas	6,061,006					97,453.50	96,743.96		3.1	5.0	427,855.00	34.6	5,536,407.04
Utah	1,048,677					418,165.45	407,005.45		40.5	11.3	114,539.25	3.1	527,132.30
Vermont	470,628					67,791.23	66,772.62		4.5	2.2	182,872.63	5.8	220,982.75
Virginia	1,854,189					37,864.96	29,104.47	8,760.49	17.2	1.9	730,589.25	7.2	1,094,495.28
Washington	1,877,571					103,208.31	103,208.31		20.3	4.1	206,258.11	4.3	1,568,104.58
West Virginia	1,342,270					39,929.76	39,929.76		35.1	.4	142,263.00	2.2	1,160,077.24
Wisconsin	2,431,220					709,629.31	709,629.31		29.3	19.8			1,721,590.69
Wyoming	1,125,332					179,583.12	179,083.12		2.2	2.9	40,628.77	2.5	905,620.11
District of Columbia	1,151,081					197,493.65	197,493.65		65.8	1.1	809,275.00	3.5	144,312.35
Hawaii													
TOTALS	113,515,642	3,656.08	3,656.08		1.3	11,580,989.12	11,277,994.03	131,683.78	18.5	241.2	14,015,360.12	254.2	88,218,631.77

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS III—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF OCTOBER 31,1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	Public works funds	Mileage	Estimated total cost	Public works funds allotted	Percentage completed	Mileage	Public works funds allotted	Mileage		
	$	$	$		$	$			$		$	
Alabama	2,092,533								85,000.00	13.5	2,007,533.00	Alabama
Arizona	625,435										625,435.00	Arizona
Arkansas	1,687,084										1,687,084.00	Arkansas
California	3,901,838				98,864.81	85,117.33	1.2	1.1	344,422.34	10.0	3,472,298.33	California
Colorado	1,718,632				245,000.00	245,000.00	4.5	61.7	50,000.00	12.8	1,423,632.00	Colorado
Connecticut	659,120								211,882.58	3.1	447,237.42	Connecticut
Delaware	454,772										454,772.00	Delaware
Florida	1,307,958				22,500.00	22,500.00	4.4	3.1	572,320.03	22.6	713,137.97	Florida
Georgia	2,320,973										2,320,973.00	Georgia
Idaho	1,121,562				548,089.73	547,177.47	12.6	90.8	80,546.18	3.1	493,838.35	Idaho
Illinois	6,262,223				22,161.15	22,161.15		3.6	29,482.33	3.9	6,210,579.52	Illinois
Indiana	501,892										501,892.00	Indiana
Iowa	2,212,245				515,923.71	491,900.00	3.9	25.6	211,700.00	31.2	1,508,645.00	Iowa
Kansas	2,522,401								417,566.37	35.5	2,104,834.63	Kansas
Kentucky	1,379,340				55,267.35	55,267.35		3.0	1,205,854.04	155.2	618,218.61	Kentucky
Louisiana	1,457,148								352,994.46	8.4	1,104,153.54	Louisiana
Maine	842,479				816,400.00	816,400.00	73.0	93.6	25,336.92	.5	742.08	Maine
Maryland	891,132										891,132.00	Maryland
Massachusetts	527,768				367,127.97	367,127.97	22.9	11.5			160,640.03	Massachusetts
Michigan	3,184,057				20,500.00	20,500.00		.7	39,000.00	5.0	3,124,557.00	Michigan
Minnesota	2,131,314				496,763.45	496,763.45	5.8	71.4	971,582.02	140.9	662,968.53	Minnesota
Mississippi	1,744,669				435,000.00	435,000.00	.7	54.6			1,309,669.00	Mississippi
Missouri	3,045,076				179,520.22	179,520.22		24.5	847,458.55	184.1	2,018,097.23	Missouri
Montana	1,859,937								67,669.25	1.2	1,792,267.75	Montana
Nebraska	1,957,240				379,485.60	377,623.28	1.6	113.6	894,386.09	147.1	685,230.63	Nebraska
Nevada	1,136,479				196,097.57	196,097.57	4.6	14.4	322,364.55	29.9	618,016.88	Nevada
New Hampshire	477,460				250,326.44	250,326.44	3.6	9.2	15,137.24	.5	211,996.32	New Hampshire
New Jersey	63,460								55,753.07	.5	7,706.93	New Jersey
New Mexico	1,448,234										1,448,234.00	New Mexico
New York	3,662,137				2,585,123.19	2,581,423.19	11.3	61.2	783,980.00	15.5	296,733.81	New York
North Carolina	2,380,573				77,758.94	72,477.17	41.4	9.1	810,107.85	61.3	1,497,987.98	North Carolina
North Dakota	1,451,112								46,035.01	10.4	1,405,076.99	North Dakota
Ohio	3,871,148				256,500.00	151,064.90	6.6	1.2	799,500.00	17.2	2,920,583.10	Ohio
Oklahoma	2,304,199										2,304,199.00	Oklahoma
Oregon	1,526,724								344,568.18	58.4	1,182,155.82	Oregon
Pennsylvania	7,716,975				2,048,250.96	2,046,860.00		214.3	3,714,262.00	354.5	1,955,853.00	Pennsylvania
Rhode Island	499,677										499,677.00	Rhode Island
South Carolina	1,364,791				179,000.00	179,000.00		31.5	805,186.38	101.1	380,604.62	South Carolina
South Dakota	1,502,870								12,525.42	3.8	1,490,344.58	South Dakota
Tennessee	2,123,155								449,061.76	56.1	1,674,093.24	Tennessee
Texas	6,061,006				397,518.53	343,700.00	5.8	46.3	1,826,736.09	340.2	3,890,569.91	Texas
Utah	1,048,677				188,701.11	188,701.11	13.8	35.4	233,000.16	11.1	626,975.73	Utah
Vermont	465,026				45,086.38	35,469.13	5.6	4.7	148,098.43	10.8	281,458.44	Vermont
Virginia	1,854,189				175,241.73	175,241.73	12.0	39.5	632,765.93	51.3	1,046,181.34	Virginia
Washington	1,180,362				301,411.30	301,410.70	7.0	23.6	325,400.78	21.0	553,550.52	Washington
West Virginia	1,118,559								231,294.90	10.4	887,264.10	West Virginia
Wisconsin	2,431,220				870,301.81	831,286.53	13.6	60.9	201,965.45	6.7	1,397,968.02	Wisconsin
Wyoming	1,125,332				80,000.00	80,000.00	12.5	13.7	166,759.26	18.1	878,572.74	Wyoming
District of Columbia	767,388				331,406.47	331,406.47	28.1	3.4	140,594.41		295,387.12	District of Columbia
Hawaii	187,106										187,106.00	Hawaii
TOTALS	94,676,687				12,185,328.72	11,926,523.16	12.4	1,127.2	18,472,298.03	1,957.2	64,277,865.81	TOTALS

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

SUMMARY OF CLASSES I, II, AND III

AS OF OCTOBER 31,1933

STATE	TOTAL APPORTIONMENT OF PUBLIC WORKS FUNDS	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR ALL NEW PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	8,370,133					1,019,052.26	560,478.75	458,573.51	3.9	38.1	1,680,390.12	174.4	6,129,264.13
Arizona	5,211,960					1,748,601.37	1,746,447.13		7.8	110.6	787,782.12	59.9	2,677,730.75
Arkansas	6,748,335					538,517.05	278,053.27	260,463.78	1.4	3.6	850,346.66	54.0	5,619,935.07
California	15,607,354					4,920,116.59	4,217,809.75		2.2	139.0	2,216,287.30	90.5	9,173,256.95
Colorado	6,874,530					1,370,195.85	1,370,195.85		27.9	118.8	768,756.66	39.0	4,735,577.49
Connecticut	2,865,740					572,131.02	572,131.02		1.7	6.1	510,099.11	5.2	1,783,509.87
Delaware	1,819,088					584,947.70	584,947.70		28.5	21.7	360,520.50	7.5	873,619.80
Florida	5,231,834					1,542,824.85	858,678.65	684,146.20	10.5	56.8	1,461,049.76	58.3	2,912,105.59
Georgia	10,091,185										594,989.10	23.5	9,496,195.90
Idaho	4,486,249					2,079,744.03	2,075,223.55		16.4	210.3	200,391.90	17.7	2,210,633.55
Illinois	17,570,770					22,161.15	22,161.15			3.6	259,342.23	9.8	17,289,266.62
Indiana	10,037,843					1,174,800.00	1,174,800.00		6.6	46.4	1,243,714.19	35.3	7,619,328.81
Iowa	10,055,660					4,737,704.23	4,463,750.00		27.7	198.4	2,004,600.00	124.0	3,587,310.00
Kansas	10,089,604					394,870.61	394,870.61		6.3	49.1	2,990,255.24	311.8	6,704,478.15
Kentucky	7,517,359					1,177,169.61	1,177,168.61		.3	85.9	2,198,097.07	230.1	4,142,093.32
Louisiana	5,828,591					753,005.33	753,005.33		16.6	22.5	1,708,291.28	40.3	3,367,294.39
Maine	3,369,917					2,028,710.94	2,028,710.94		44.6	130.1	333,175.89	7.2	1,008,030.17
Maryland	3,564,527					173,094.69	159,295.94		7.5	5.4	674,613.95	10.0	2,730,617.11
Massachusetts	6,597,100					1,133,523.06	902,576.85	219,984.24	24.7	29.7	1,621,137.30	21.5	4,073,385.85
Michigan	12,736,227					1,363,470.00	1,363,470.00		39.0	76.1	739,850.00	49.8	10,632,907.00
Minnesota	10,656,569					3,538,166.65	3,538,166.63		32.8	479.0	1,596,773.85	187.0	5,521,628.52
Mississippi	6,978,675					830,923.59	670,028.93	160,894.66	1.8	74.9	280,857.36	27.2	6,027,788.71
Missouri	12,180,306					2,362,248.81	2,223,577.48		5.4	85.2	2,905,273.95	263.8	7,051,454.57
Montana	7,439,748					3,491,762.75	3,311,762.75	100,000.00	9.4	259.6	542,899.49	25.4	3,585,085.76
Nebraska	7,828,961	46,788.08	42,867.02		19.5	3,540,561.95	2,813,788.51	108,538.00	24.7	359.2	1,834,627.95	218.2	3,137,677.52
Nevada	4,545,917					774,025.81	774,025.81		19.4	97.2	943,680.72	109.6	2,828,210.47
New Hampshire	1,909,839					716,543.49	676,509.94		11.2	17.4	81,451.23	2.0	1,151,877.83
New Jersey	6,346,039					999,228.43	999,228.43		14.1	11.5	1,434,601.15	8.7	3,912,209.42
New Mexico	5,792,935					1,776,445.05	1,776,445.05		17.4	174.2	952,957.20	68.1	3,063,532.75
New York	22,330,101					13,438,472.19	13,130,013.19	300,000.00	8.4	261.6	5,714,480.00	54.8	3,485,607.81
North Carolina	9,522,293					962,532.11	555,452.48	401,797.86	30.1	108.4	1,968,118.20	226.5	6,598,722.32
North Dakota	5,804,448					888,082.13	888,082.13		32.5	364.1	1,436,383.65	360.1	3,479,982.22
Ohio	15,484,592					4,386,730.00	4,191,319.90		22.4	98.3	4,278,490.00	86.4	7,014,782.10
Oklahoma	9,216,798					1,800,851.56	1,800,851.56		4.5	116.1	2,004,191.59	104.7	5,411,754.85
Oregon	6,106,896					1,979,979.20	1,931,679.70		17.1	110.6	1,459,620.58	122.4	2,715,595.42
Pennsylvania	18,891,004					2,712,853.72	2,711,462.76			231.0	7,647,198.67	454.3	8,532,342.57
Rhode Island	1,998,708					952,043.50	952,043.50		14.7	19.5	42,490.08	.7	1,004,174.42
South Carolina	5,459,165					924,002.43	920,867.80	3,134.63	4.1	125.0	1,388,114.46	167.2	3,150,182.74
South Dakota	6,011,479					1,231,298.38	1,196,368.78	34,929.60	35.3	188.8	1,021,584.94	115.5	3,793,525.28
Tennessee	8,492,619					815,341.50	524,915.96	290,425.54	12.2	15.9	1,177,013.29	99.5	6,790,689.75
Texas	24,244,024					3,454,791.01	2,694,060.61		8.4	337.6	4,945,781.18	704.8	16,604,182.21
Utah	4,194,708					2,055,251.52	2,044,091.52		47.4	210.3	520,338.76	17.3	1,630,277.42
Vermont	1,867,573					724,556.09	713,920.23		12.2	35.9	422,915.35	25.8	730,735.92
Virginia	7,416,757					811,861.61	785,918.25	25,943.36	8.9	67.2	2,616,104.43	108.8	4,014,734.32
Washington	6,115,867					1,558,065.91	1,554,708.49	3,355.82	20.8	78.6	1,214,248.74	50.6	3,346,909.77
West Virginia	4,474,234					1,361,236.08	1,361,236.08		22.4	59.0	833,410.95	26.3	2,279,586.97
Wisconsin	9,724,881					4,144,378.13	4,083,862.85	21,500.00	29.1	189.8	624,710.87	36.0	5,016,307.28
Wyoming	4,501,327					2,032,606.07	1,888,383.12	60,000.00	19.2	371.7	532,274.02	96.6	2,080,669.86
District of Columbia	1,918,469					528,900.12	528,900.12		42.2	4.5	949,869.41	3.5	439,699.47
Hawaii	1,871,062					87,001.76	43,500.88	43,500.88		5.1	157,010.70	5.1	1,670,550.42
TOTALS	394,000,000	46,788.08	42,867.02		19.5	92,215,381.19	85,988,948.84	3,177,189.08	17.1	5,909.7	74,731,163.95	5,147.2	233,237,020.19

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 14, NO. 10 DECEMBER 1933

RAISING PAVEMENT BY MUD JACKING

For sale by the Superintendent of Documents, Washington, D.C. - - - - - - - - - - - - - - See Page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 10 December 193

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data a obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is don the conclusions formulated must be considered as specifically pertinent only to described conditions

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

Laboratory Tests Assist in the Selection of Materials Suitable for Use in Mud Jack Operations

Reported by A. M. WINTERMYER, Assistant Highway Engineer, Division of Tests, United States Bureau of Public Roads

IN THE routine procedure for testing subgrade soils which has been described previously,[1] it was desired to include tests sufficient in number and varied enough in character to identify all of the various soils apt to be encountered in highway construction in all parts of the country and place each soil in the proper subgrade group as indicated by its physical characteristics.

Much of the soil testing is now done for one of the following purposes: (*a*) Determining the suitability of soil or soil material for some particular use such as binder in sand-clay and gravel roads, filler in bituminous road surfacing or soil for mud jack operations; (*b*) distinguishing between the good and the undesirable varieties of individual groups of surfacing materials such as limerock, caliche, shale, etc.; (*c*) identifying the properties of materials such as rock powders, diatomaceous earths, bentonites, etc., which, as admixtures may assist in the stabilization of both subgrades and soil road surfaces.

In testing materials for a special use the tests may be limited to those required to disclose the particular characteristics essential for the particular use. Studies to determine testing procedure consist of three distinct steps as follows:

(*a*) Analysis to determine what is to be required of the material for the use intended.

(*b*) Determination of the dominating characteristics satisfying these requirements.

(*c*) Selection of the particular tests best suited to furnish the desired information on these dominating characteristics.

The following suggested procedure in the study of soils for use in mud jack operations illustrates the essential features of an investigation of the suitability of soils for special purposes.

MATERIALS FOR MUD JACKING MUST HAVE CERTAIN CHARACTERISTICS

A satisfactory mixture of soil, portland cement, and water for use in raising settled areas of road surfaces by mud jacking (see fig. 1) must be of such a character that it can be readily forced through the mud jack. It must possess qualities which will enable it to spread freely over the subgrade as the separation between the road surface and subgrade increases during the pumping operation, and it must prevent appreciable settlement of the raised area of pavement after the pumping operation.

FIGURE 1.—LOW PLACE IN CONCRETE PAVEMENT BEING RAISED TO GRADE BY MUD JACKING.

The mixture must be sufficiently plastic to permit its being forced readily through the pump and the hose connecting the pump with the opening in the pavement. Such a state of fluidity may be obtained with a variety of mixtures ranging from the smoothest cohesive pastes to those which have an appreciable degree of harshness. All of the mixtures which are fluid enough to perform satisfactorily in the pump, however, may not be capable of spreading to the desired extent over the subgrade. This is because the conditions controlling the spread of the mixture over the subgrade differ somewhat from those controlling the flow of the mixture through the pump and hose.

In passing through the pump and hose the mixture is confined to a definite channel of comparatively large size and only the proper combination of pump pressure and fluidity of mixture are required. When the paste reaches the subgrade, however, its flow is not restricted to any particular cross section. It is free to form in layers whose ratios of thickness to area of distribution depend to a very considerable extent on the frictional

[1] Reports on Subgrade Soil Studies, reprinted from Public Roads, vol. 12, nos. 4, 5, 7, and 8.

resistance of the subgrade and the under face of the road slab which bound the shallow openings penetrated by the paste.

The smaller this ratio, the more uniform is the slab support both during and subsequent to the pumping operation. As the ratio increases there is an increasing tendency of mixtures to accumulate under certain slab areas, especially at the location where the paste is forced through the pavement.

The frictional resistance depends not only on the characteristics of the subgrade and the under side of the road slab but also upon the harshness of the paste.

The more cohesive mixtures spread most readily in a thin layer over a large area. With increasing harshness of mixtures there is an increasing tendency for the paste to form in accumulations leaving unsupported areas which are productive of cracking.

Harshness of mixture affects the frictional resistance of pump and hose but within the range of mixtures having the desired fluidity, the difference in harshness has but little practical significance, insofar as the pumping operation is concerned. Mixtures not sufficiently fluid are objectionable primarily because of their tendency to jam in the pump but they may also contribute to unsatisfactory distribution under the slab. Harsh mixtures are objectionable primarily because of their tendency to accumulate under limited slab areas and it is possible for them to be harsh enough to jam in the pump. Mixtures having harshness sufficient to jam in the jack are easily recognized and should be discarded. Experience indicates that a mixture considered by an experienced operator to be sufficiently fluid for use in a pump will spread satisfactorily over the subgrade.

Raised areas of pavement may settle after the pumping ceases as a result of several causes. The weight of the pavement may cause the viscous paste to flow out as soon as the pump stops or the paste may shrink on loss of moisture. The paste or soil mixture must be stable enough to support the pavement and loads produced by traffic immediately after pumping ceases and must resist shrinking upon loss of moisture.

The essential qualifications of a mixture for use in mud jacking can be stated as follows:

1. The paste should be fluid enough to be readily pumped.
2. It should be free enough from harshness to spread to the desired extent over the subgrade.
3. The paste, even in the high state of fluidity required for satisfactory performance in the pump, must not be subject to detrimental shrinkage upon loss of moisture.
4. The paste, after being properly placed, must change rapidly from a highly fluid, smooth-flowing state to one which is stable enough to resist lateral flow under considerable pressure.

CHARACTERISTICS OF SUITABLE MATERIALS STATED

The fluid paste-forming properties of a mixture are furnished by the finer soil particles which comprise the silt and the clay fractions. Sand increases the stability of the mixture and reduces the shrinkage upon drying. The harshness of the mix is furnished by the sand fraction. Generally, the larger the sand particles or the higher the sand content, the harsher will be the soil paste.

A sufficient amount of sand to furnish all of the stability required at the conclusion of the pumping would introduce harshness so great that the mixture could not be forced through the pump. It is not possible to use enough sand to eliminate all detrimental shrinkage.

A rapid increase in stability and elimination of detrimental shrinkage must be obtained with an admixture capable of permanently stabilizing the mixture to the desired extent. Portland cement, when added in sufficient amount, serves well for this purpose.

Studies of topsoil roads indicate that the presence of coarse sand particles (those retained on the no. 60 sieve) contributes to the hardness of a soil surface and increases its resistance to abrasion. When abrasive action is absent, fine sand particles (those passing the no. 60 sieve) may be just as efficient as coarse sand in stabilizing the soil structure. The harshness caused by fine sand is materially less than that caused by coarse sand. Therefore, fine sand seems better than the coarse sand for use in mud jack operations.

A suitable soil must be of such character that it will slake readily in water to form a mixture of uniform consistency. It must be free from glue-like colloids which do not soften readily when mixed with water and are productive of objectionable packing of the mixture in the mud pump.

Soil for use in mud jacking should consist of fine sand, silt, and clay in such proportions that the paste formed from the silt and clay will completely encompass all of the sand particles so as to eliminate detrimental harshness of the mixture. The silt and clay fraction should be free of gluey colloids productive of high-water capacity, high plasticity, and excessive shrinkage and, in addition, should be of the inactive character required to form a paste of the desired consistency and smoothness with minimum additions of water.

Generally, the greater the shrinkage properties of the soil, the larger will be the cement admixture required to prevent excessive shrinking. Admixture of cement sufficient to accomplish this purpose is considered sufficient to prevent detrimental flow of the paste after placing.

TESTS SHOULD BE SELECTED WHICH WILL DISCLOSE DESIRED CHARACTERISTICS

The tests selected for studying the properties of the mixtures under discussion should furnish information on the following:

(*a*) The grading of the materials.
(*b*) The shrinkage properties of the mixture.
(*c*) The character of the fine fraction of the soil.
(*d*) The fluidity of the mixture.
(*e*) The harshness of the mixture.

The mechanical analysis, the shrinkage limit, the liquid limit, and the preparation of flow curves of soils, are suggested as complying with these requirements.

Complete information on mechanical analysis, the shrinkage limit, the liquid limit, and flow curves has been given in previous reports.[2] Therefore, the tests are referred to but briefly in this report. The essential features of the mechanical analysis are well understood and require no discussion. Results of mechanical analyses are shown graphically in figure 2.

The shrinkage limit is defined as the moisture content at which further evaporation does not produce further decrease in the volume of the soil. As water evaporates from a soil mixture, capillary tension acting like a taut skin on the surface of the soil mass

[2] Reports on subgrade soil studies, reprinted from Public Roads vol 12, nos. 4, 5, 7, and 8, and Research on the Atterberg Limit of Soils, Public Roads, vol. 13, no. 8.

FIGURE 2.—RESULTS OF MECHANICAL ANALYSIS OF SOIL SAMPLES SHOWING CUMULATIVE PERCENTAGES BY WEIGHT.

gradually draws the soil particles closer together until the shrinkage limit is reached. At this moisture content the resistance of the soil to further compression just equals the force exerted by the evaporating moisture. Below the shrinkage limit the resistance to compression exceeds the compressing force. Therefore, further evaporation below the shrinkage limit is not productive of further decrease in the volume of the soil mass. Soil mixtures used in mud jacking are not apt to shrink appreciably when their shrinkage limits are equal to or exceed the moisture content required to give the mixture the desired fluidity.

The liquid limit is the moisture content at which a groove (see fig. 3) separating two parts of a divided soil cake is just closed by a given number of shocks. This test is indicative of a particular degree of stability or consistency of the soil. In the routine or hand method of test, water or soil powder as required is added to the soil sample until the desired consistency, that requiring 10 shocks to close the soil groove, is obtained.

In the mechanical method (see fig. 4), the number of shocks required to close the groove with the soil at several consistencies are determined and plotted against the corresponding moisture contents to form "flow" curves. The liquid limit as determined by the hand method is the same as the moisture content corresponding to 25 shocks in the mechanical method. The number of shocks required to close the soil groove is a measure of the stability of the mixture and the flow curves, which are straight lines on semilog plots, disclose the relation between the moisture content and the degree of fluidity or flowability of the mixtures (see fig. 5).

It follows from this that the number of shocks or fraction of a shock corresponding to a particular moisture content on the flow curve extended becomes a quantitative measure of the consistency of the mixture at this moisture content. Therefore, all soils, at the moisture content corresponding to a given number of blows (including fractions of a blow) on the flow curves, should have the same degree of fluidity. Consequently, the number of blows indicative of the fluidity of mixtures required for mud-jack purposes as determined by tests on a comparatively few soils may serve as an index with which the required water content of soils generally may be determined.

The ratio of the liquid limit to the clay content discloses the relative activity of the clay fraction. Liquid limits approximately equal to the clay content indicate the presence of inactive fine particles capable of forming pastes with additions of relatively small amounts of water. The more the liquid limit exceeds the clay content the more the presence of undesirable gluey colloids is indicated.

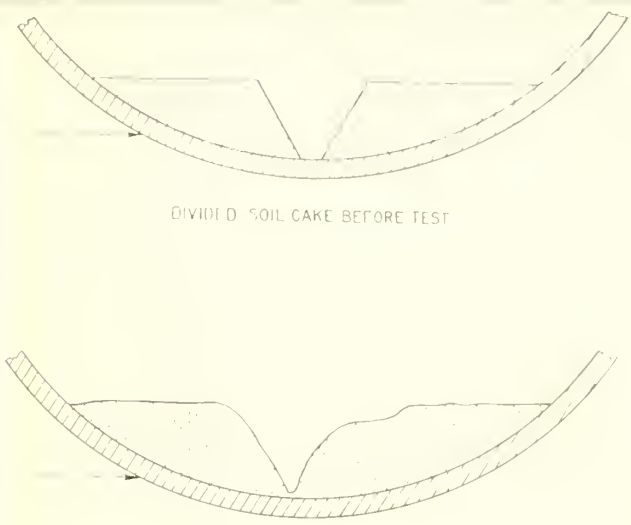

FIGURE 3.—UPPER—DIVIDED SOIL CAKE BEFORE RECEIVING 10 LIGHT BLOWS AT LOCATION INDICATED BY ARROW. LOWER LATERAL FLOW (SHEAR FAILURE) OF CAKE DURING TEST.

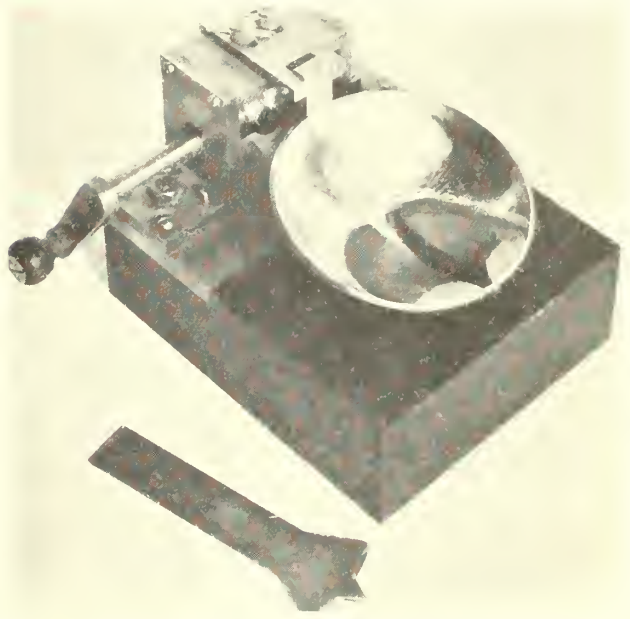

FIGURE 4.—MECHANICAL DEVICE FOR MAKING THE LIQUID LIMIT TEST.

The ratio of the shrinkage limit to the liquid limit indicates the shrinkage properties of the soil. The smaller the shrinkage limit as compared with the liquid limit the greater will be the tendency for the soil to shrink and consequently the greater will be the amount of cement required to prevent the shrinkage from becoming objectionable.

LABORATORY PROCEDURE ILLUSTRATED

Table 1 gives the results of laboratory tests of four soils which have been used in mud-jack operations with various results. Sample no. S-6361 is a Virginia soil used in mud jacking on the Mount Vernon Memorial Highway. This soil was selected by an experienced operator because of its ability to form a paste of the desired consistency and smoothness. The other three

TABLE 1.—*Results of laboratory tests on soils used in mud jacking*

Sample no.	Mechanical analysis, percentage by weight					Liquid limit	Shrinkage limit
	Passing no. 40 sieve	Coarse sand 2 to 0.25 mm	Fine sand 0.25 to 0.05 mm	Silt 0.05 to 0.005 mm	Clay smaller than 0.005 mm		
S-6361	99	5	27	13	25	26	21
S-7100	98	9	27	35	29	33	15
S-7462	99	3	10	29	58	72	9
S-7463	99	5	63	24	8	16	(1)

(1) Pat did not shrink in test.

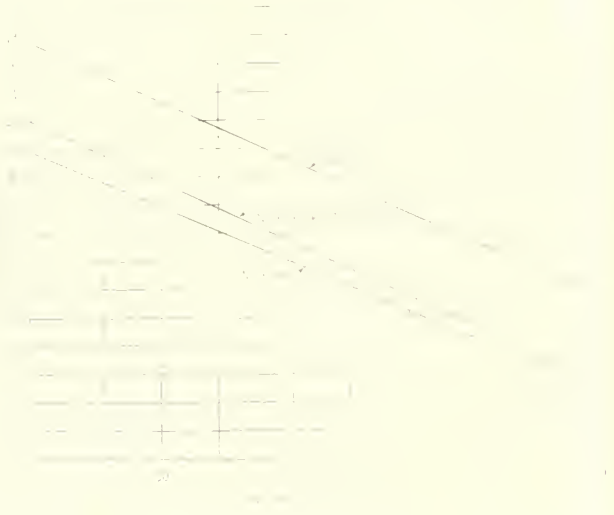

FIGURE 5.—FLOW CURVES OF SOILS TESTED FOR SUITABILITY FOR MUD JACKING.

samples represent Texas soils, the performance of which is reported to have been as follows:

That represented by sample no. S-7100 formed a mixture which worked well in the pump. Sample no. S-7462 has a high clay content and this soil could not be formed into a smooth paste and jammed in the pump. Sample no. S-7463 has a high sand content and formed a mixture so harsh that it "packed" both in the pump and under the pavement.

The absence of coarse sand in an appreciable amount is the only conspicuous feature common to all four of the soils. Samples no. S-6361 and no. S-7100 have somewhat similar gradings and their flow curves (fig. 4) are approximately parallel. Sample no S-7463 was so harsh that the mechanical device for liquid limit determination could not be used. A liquid limit of 16 was determined by the hand method. The presence of gluey colloids in sample no. S-7462 made it difficult to obtain uniform mixtures of water with this soil even in the laboratory which made it necessary to determine the liquid limit of this soil by the hand method.

The soil represented by sample no. S-6361 seems particularly well suited for mud jacking and its essential characteristics may serve as a guide in studies of soils for this purpose. This soil is representative of well graded and thoroughly weathered topsoils with little or no material retained on the no. 40 sieve. It is stable in road surfaces in both wet and dry weather but erodes badly in cut or fill faces. The fine sand (27 percent) furnishes stability without producing harshness of mix. The clay (25 percent), is relatively inactive. This is disclosed by the low liquid limit of 26 as compared with a typical liquid limit of about 36 for average soils containing 25 percent of clay. The property which causes this low liquid limit is responsible for the erosion referred to above and also for relatively low shrinkage properties. The grading of this soil is uniform and the mechanical analysis, when plotted as shown in figure 2, approximates a straight line. (The equation of this line is $P = 38 (\log d + 3)$ in which P equals the percentage of particles smaller than the particle size d.)

Its liquid limit is practically equal to the clay content and its shrinkage limit is about 81 percent of the liquid limit. This latter value is significant because, on the average, shrinkage limits equal to or greater than about 75 percent of the liquid limits are indicative of inactive soils in which shrinkage is not important. The shrinkage limit is somewhat smaller than the liquid limit, indicating the presence of cohesive paste-forming particles. At present it is not known whether the slope of the flow curves, figure 5, has any special significance.

The moisture content required to furnish the necessary fluidity of this soil, as determined by trial in the mud-jack operations, was found to be about 40 percent without cement admixture and about 42 percent with the cement admixture. Figure 5 shows that these moisture contents furnish a consistency corresponding to 0.1 blow on the flow curve extended.

The effect of cement admixtures in raising the shrinkage limit is shown in table 2. In these determinations, the soil and cement were mixed with 42 percent of water, molded into pats, and then allowed to dry to constant weight. No shrinkage of the pat occurred when the addition of cement became equal to 7 percent of the weight of the soil. Therefore it is safe to assume that the mixture in service will not shrink when containing cement in this amount.

TABLE 2.—*Effect of cement admixtures upon the shrinkage limit of the soil represented by sample no. S-6361*

Cement admixture, percent by weight	Shrinkage limit
1	29
3	31
5	38
7	(1)

(1) Pat did not shrink in test

Upon this basis, the suggested mixture would contain about 100 pounds of dry soil, 7 pounds of portland cement, and 45 pounds or about 5½ gallons of water.

Sample no. S-7100 with a shrinkage limit of 15 and a required moisture content of about 48 percent (value corresponding to 0.1 in fig. 4) will shrink more than soil no. S-6361 with a shrinkage limit of 21 and a moisture content of mixture of about 42 percent. More cement would have to be added to S-7100 than to S-6361 to entirely eliminate the possibility of shrinkage. Since soils S-7463 and S-7462 could not be formed into pastes sufficiently fluid to be tested in the liquid limit device it is quite evident that they are not suitable for mud jack operations. In addition the low shrinkage limit of 9 for soil S-7462 indicates the presence of objectionable colloids in dominating amounts. The absence of shrinkage in the test of soil S-7463 suggests the absence of the cohesive paste-forming fine soil particles.

(Continued on page 194)

Analytical Tools for Judging Results of Structural Tests of Concrete Pavements

By H. M. WESTERGAARD, Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

INFORMATION concerning stresses in pavements is needed for the purpose of establishing policies of design, for the purpose of imposing proper limits on the loads that may be transported, and for the purpose of apportioning the cost of pavements to the different kinds of traffic, with a view toward rational taxation of vehicles of various weights. The last reason has been a particular incentive in conducting structural studies of pavements during recent years.

Knowledge of the structural behavior of concrete pavements depends on tests and theory. Results of a theory were reported in a paper in 1926.[1] This theory, in the main, was supported by the experimental data available at the time. It supplies answers to the question, what deflections and stresses may be expected when the subgrade is within a range of normal conditions, without unusual heaving and irregularity. This theory was used by the Bureau of Public Roads in planning a series of tests of pavement slabs. The tests were carried out by the Bureau of Public Roads at Arlington, Va., in 1932. The theory, having served in planning the tests, will naturally also furnish tools for examining the results of the tests. The tests suggest certain adaptations of the theory. There will be needed some restatements of analytical results, and some supplementing and modification of the theory.

MEASURES OF STIFFNESS IN THE ORIGINAL THEORY

In the formulas used to demonstrate the original theory, as presented in 1926, various quantities appear as measures of stiffness, which are assumed to be constant in each particular case. The modulus of elasticity, E, and Poisson's ratio, μ, of the concrete were introduced as constants. Furthermore, the resistance of the subgrade was expressed by a quantity, k, which is called the modulus of subgrade reaction. This modulus is defined as the reaction per unit of area per unit of deflection, and it may be measured in pounds per square inch per inch; or, pounds per cubic inch. It was admitted that this modulus could not be expected to be an actual constant of the material, expressing properties of the subgrade independently of the slab. But it was found that a rather wide variation of k led to relatively small variations of the stresses, and consequently the modulus k was introduced as a quantity which might be assumed to be constant for a given combination of slab, subgrade, and position of the load.

From E, μ, k, and the thickness of the slab, h, a further quantity was obtained, a distance l, which is called the radius of relative stiffness. This distance appears in a number of the formulas and is defined by the equation,

$$l^4 = \frac{Eh^3}{12(1-\mu^2)k} \qquad (1)$$

[1] Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard, Public Roads, vol. 7, no. 2, April 1926

FORMULAS FOR THE STRESSES

In the previous paper approximate formulas for stresses in the interior of the area and at the edge (equations 11 and 12) were stated for the special case in which $E = 3{,}000{,}000$ pounds per square inch and $\mu = 0.15$. To make the theory usable in interpreting results of new tests, it is desirable to restate these formulas, so that they may be used for any values of E and μ. A third approximate formula, for the stress at a corner (equation 7), was stated for any values of E and μ.

The following notation is used:

P = load.

a = (1) radius of a small circle over the area of which P is assumed to be distributed uniformly when the load is at a considerable distance from the edges; (2) radius of a small semicircle over the area of which P is assumed to be distributed uniformly when the load is at the edge, the center of the semicircle being at the edge; (3) distance of the point of application of the resultant of the load from each of the two edges at a rectangular corner when P is close to this corner (the distance from the corner being $a_1 = a\sqrt{2}$).

σ_i = maximum tensile stress at the bottom of the slab directly under the load P, when P is at a considerable distance from the edges.

σ_e = maximum tensile stress at the bottom of the slab at the edge due to a load at the edge.

σ_c = maximum tensile stress at the top of the slab in a diagonal direction due to a load at a corner.

b = radius depending on a and h.

The radius b may be computed by the following approximate formulas (equation 8 in the previous paper):

$$b = \sqrt{1.6a^2 + h^2} - 0.675h \text{ when } a < 1.724h \qquad (2)$$

$$b = a \text{ when } a > 1.724h \qquad (3)$$

The set of formulas for stresses becomes:

Stress in the interior of the area, at the bottom:

$$\sigma_i = 0.275(1+\mu)\frac{P}{h^2}\log_{10}\left(\frac{Eh^3}{kb^4}\right) \qquad (4)$$

Stress at the edge, at the bottom:

$$\sigma_e = 0.529(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right) - 0.71\right] \qquad (5)$$

Stress at the corner, at the top:

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{12(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right] \qquad (6)$$

With $E = 3{,}000{,}000$ pounds per square inch and $\mu = 0.15$ equations 4 and 5 give the same results as equations 11 and 12 in the previous paper. Equation 6 is a transcription of equation 7 in the previous paper

NEW COEFFICIENTS PROPOSED

It is proposed to introduce a new coefficient, K, defined by the relation

$$K = kl \quad (7)$$

The coefficient K, like k, will be a measure of the resistance of the subgrade. The reason for the proposal is the expectation that K will be less dependent on the stiffness of the slab than is k. If the depressions of circular bearing blocks under a moderate load are inversely proportional to the diameter, then the product of the corresponding k and the diameter would be constant. Correspondingly, in case of the slab, it is at least a possibility that the product of the most plausible value of k times the linear dimension l will be fairly constant for a given condition of the subgrade. It appears desirable, therefore, to supplement the formulas in terms of k by a set of equivalent formulas in terms of K. The truth may lie between the two extreme cases of a constant k and a constant K.

It is suggested that K be called the coefficient of subgrade stiffness. It is noted that K may be measured in pounds per square inch. For example, $k = 50$ lb. in.$^{-3}$ and $l = 30$ in. give $K = 1{,}500$ lb. in.$^{-2}$

The following relations are derived from equations 1 and 7:

$$l^3 = \frac{Eh^3}{12(1-\mu^2)K} \quad (8)$$

$$K = \frac{Eh^3}{12(1-\mu^2)l^3} \quad (9)$$

$$\frac{K^4}{k^3} = \frac{Eh^3}{12(1-\mu^2)} \quad (10)$$

It is proposed furthermore, to introduce a coefficient D, defined as

$$D = kl^2 \quad (11)$$

or

$$D = Kl \quad (12)$$

and to call this coefficient the deflection modulus of the pavement. The reason for this proposal is the consistent occurrence of the quantity kl^2 in the formulas for the deflections. In equations and diagrams in the previous paper the deflections are stated in the general form:

$$z = c\frac{P}{kl^2} \quad (13)$$

where c is a pure number in each case. For example, when P is applied at the interior of the area of the slab, and the deflection z is measured directly under the load, equation 14 in the previous paper gives $c = \frac{1}{8}$. In this case, if the pavement is considered as a spring, $8kl^2$ is the modulus of the spring. With D introduced, the deflections will be restated as

$$z = c\frac{P}{D} \quad (14)$$

The following further relations involving D are derived from the preceding equations:

$$D = \sqrt{\frac{Eh^3k}{12(1-\mu^2)}} = \sqrt[3]{\frac{Eh^3K^2}{12(1-\mu^2)}} \quad (15)$$

$$k = \frac{12(1-\mu^2)D^2}{Eh^3} \quad (16)$$

$$K = \sqrt{\frac{12(1-\mu^2)D^3}{Eh^3}} \quad (17)$$

$$E = \frac{12(1-\mu^2)D^2}{h^3} \quad (18)$$

RESTATEMENT OF THE FORMULAS FOR THE STRESSES IN TERMS OF THE NEW COEFFICIENT OF SUBGRADE STIFFNESS

Since $1-\mu^2$ does not vary a great deal, this quantity may be computed as if μ were constant, equal to 0.15, which gives $1-\mu^2 = 0.9775$. Then equations 4 and 5 may be restated as follows by use of equation 10:

Stress in the interior of the area, at the bottom:

$$\sigma_i = 1.1(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{h}{b}\right)+\frac{1}{3}\log_{10}\left(\frac{E}{K}\right)-0.089\right] \quad (19)$$

Stress at the edge, at the bottom:

$$\sigma_e = 2.117(1+0.54\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{h}{b}\right)+\frac{1}{3}\log_{10}\left(\frac{E}{K}\right)-0.2666\right] \quad (20)$$

The stress at the corner, at the top becomes, by restatement of equation 6,

$$\sigma_c = \frac{3P}{h^2}\left[1-\left(\frac{12(1-\mu^2)K}{Eh^3}\right)^{0.2}(a\sqrt{2})^{0.6}\right] \quad (21)$$

DERIVATION OF THE CONSTANTS FROM TESTS

Six quantities assumed constant under given conditions occur in the equations which have been stated: E, μ, k, K, D, and l. Since a variation of Poisson's ratio, μ, has only a relatively small influence, it is sufficient to know an approximate value of μ. There remain then the five quantities E, k, K, D, and l. If two of them can be determined to begin with by a direct examination of experimental data, then equations will be available by which the remaining three may be computed. Table 1 shows what equations are to be used.

TABLE 1.—*Key to the computations of 3 of the constants E, k, K, D, and l from the remaining 2*

Known beforehand	Computation by equations
E and k	K by (10), D by (15), l by (1).
E and K	k by (10), D by (15), l by (8).
E and D	k by (16), K by (17), l by (18) or (11) or (12).
E and l	k by (1), K by (9), D by (18) or (11) or (12).
D and l	E by (18), k by (11), K by (12).

The modulus of elasticity, E, may be obtained with a reasonable degree of certainty from tests of cylinders and control beams. The deflection modulus, D, may be obtained from observed maximum deflections by equation (14), with the values of c as given in the previous paper. The radius of relative stiffness, l, appears as a unit of the horizontal scales in the diagrams of deflections in the previous paper, and may be obtained, therefore, by comparing the diagrams of deflections

observed at a series of points with the diagrams of deflections in the previous paper. By fitting both the horizontal and the vertical scales of a theoretical diagram of deflections of a series of points to the corresponding diagram of observed deflections, so as to make the two curves coincide as nearly as possible, D and l may be determined together.

When strains have been measured, and E and μ are known approximately, the corresponding stresses may be computed from the strains. It would seem possible thereafter to compute k or K from equations 4 to 6 or 19 to 21. Inspection of these equations shows, however, that values of k and K determined in this way will be very sensitive to small variations of the stresses. It appears preferable, therefore, to assign various definite values to k or K and thereafter compare the experimental and analytical results.

In dealing with the stresses it is to be remembered that σ_i and σ_e are tensile stresses at the bottom of the slab. With small values of the radius, a, the compressive stresses at the top must be expected to show a somewhat greater concentration and therefore greater maximum values than would be found for the corresponding tensile stresses at the bottom.

SUPPLEMENTARY THEORY BASED ON REDISTRIBUTION OF THE SUBGRADE REACTIONS

This supplementary theory is limited to the case of a load applied in the interior of the area of the slab. In the original theory the reaction of the subgrade per unit of area at each point was assumed to be proportional to the deflection of the slab at that point. If the subgrade acts as a continuous body, it is to be expected that the reactions of the subgrade will be more closely concentrated around the load than are the deflections. Accordingly a set of corrections of the deflections will be introduced which may account for a redistribution of the reactions with an increase of the reactions near the load and a decrease of the reactions at greater distances from the load.

The following notation is used:

r = radial distance measured horizontally from the point of application of the resultant of the load P.

L = maximum value of r within which adjustments are made.

z = deflection according to the original theory.

z' = superimposed supplementary deflection.

$z'' = z + z'$ = resultant deflection.

z'_i, z''_i = values of z' and z'' at the point $r = 0$.

kz = reaction per unit of area according to the original theory.

q = supplementary reaction per unit of area, accounting for the deflection z'.

$kz + q$ = resultant reaction per unit of area.

q_i = value of q at $r = 0$.

ϵ'_i = strain at the bottom of the slab at $r = 0$ due to the deflection z'.

σ'_i = supplementary stress at the bottom of the slab at $r = 0$, corresponding to the deflections z'.

$\sigma''_i = \sigma_i + \sigma'_i$ = resultant stress at the bottom of the slab at $r = 0$, σ_i being defined by equation 4 or equation 19.

$Z = -\frac{z'_i}{z_i}$ = ratio of reduction of the maximum deflection.

The following formulas are proposed for the supplementary deflections:

$$z' = z'_i\left(1 - \frac{r^2}{L^2}\right)^{10} \text{ when } r < L \qquad (22)$$

$$z' = 0 \text{ when } r \geq L \qquad (23)$$

Instead of the power 10 some other power might be used, but the power 10 appears to be suitable. A plausible value of L may be $5l$.

The deflections z' are caused by the reactions q, which may be interpreted as loads taken positive upward.

According to the theory of slabs [2] the upward load per units of area, that is, the reaction, may be stated as

$$q = -\frac{Eh^3}{12(1-\mu^2)}\Delta^2 z' \qquad (24)$$

in which Δ is the operator $\frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$, x and y being rectangular coordinates. In the present case it is preferable to use the equivalent expression for Δ in terms of the radius vector r, namely,[3]

$$\Delta = \frac{d^2}{dr^2} + \frac{1}{r}\frac{d}{dr} \qquad (25)$$

Equations 22, 24, and 25 give

$$q = q_i\left(1 - 18\frac{r^2}{L^2} + 45\frac{r^4}{L^4}\right)\left(1 - \frac{r^2}{L^2}\right)^6 \qquad (26)$$

in which q_i, the value directly under the load, is

$$q_i = \frac{240Eh^3 z'_i}{(1-\mu^2)L^4} \qquad (27)$$

It is noted that the integral of q over the whole area is zero. That is, the total vertical force added to P is zero, and the reactions are merely redistributed.

The curvature of the slab in any direction, at $r = 0$, due to the supplementary deflections z', becomes

$$-\left[\frac{d^2 z'}{dr^2}\right]_{r=0} = \frac{20z'_i}{L^2} \qquad (28)$$

The corresponding strain at the bottom of the slab is

$$\epsilon'_i = \frac{10h}{L^2}z'_i \qquad (29)$$

and the corresponding stress is

$$\sigma'_i = \frac{10Eh}{(1-\mu)L^2}z'_i \qquad (30)$$

By use of equation 18 this stress is restated as

$$\sigma'_i = \frac{120(1+\mu)D}{h^2}z'_i\left(\frac{l}{L}\right)^2 \qquad (31)$$

The deflection directly under the load according to the original theory is given by equation 14 in the previous paper, and is

$$z_i = \frac{Pl^2}{8D} \qquad (32)$$

[2] See, for example, the paper, Computation of Stresses in Bridge Slabs Due to Wheel Loads, by H. M. Westergaard, Public Roads, vol. 11, no. 1, March 1930, especially equations 10, 15, and 16.

[3] See, for example, A. and L. Foppl, Drang und Zwang, vol. 1, 2d edition, 1924, p. 174, equations 85 and 86.

It will be expedient to express the supplementary deflection at the same point in terms of a ratio of reduction Z, as

$$z'_i = -Zz_i = -Z\frac{P}{sD} \quad \text{---} \quad (33)$$

Then the resultant deflection directly under the load becomes

$$z''_i = z_i + z'_i = (1-Z)\frac{P}{sD} \quad \text{---} \quad (34)$$

Equations 31 and 33 give

$$\sigma'_i = -\frac{15(1-\mu)ZP}{h^2}\left(\frac{l}{L}\right)^2 \quad (35)$$

The resultant maximum stress at the bottom of the slab is $\sigma''_i = \sigma_i + \sigma'_i$. With the stress σ_i expressed by equation 4 or 19 one finds the resultant stress,

$$\sigma''_i = 0.275(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right) - 54.54\left(\frac{l}{L}\right)^2 Z\right] \quad (36)$$

in terms of k, or

$$\sigma''_i = 1.1(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{h}{b}\right) + \frac{1}{3}\log_{10}\left(\frac{E}{K}\right) - 0.089 - 13.64\left(\frac{l}{L}\right)^2 Z\right] \quad \text{---} \quad (37)$$

in terms of K.

In interpreting the results of tests the new equations may be used as follows: Values of Z and $\frac{L}{l}$ are estimated (for example, $\frac{L}{l} = 5$). The diagram for $z'' = z + z'$ is drawn by use of Figure 4 in the previous paper and by use of equations 22 and 33. With Z and $\frac{L}{l}$ chosen within proper ranges this diagram should correspond in shape to those obtained experimentally for a series of points. A comparison of the experimental and theoretical diagrams of deflections for a series of points will lead to a determination of D and l. Then E may be computed by equation 18, and k or K by equation 11 or 12, respectively. Finally the stress at the bottom of the slab may be computed by equation 36 or 37, and the computed values may be compared with those obtained experimentally. It is noted that with a chosen value of $\frac{L}{l}$ a variation of Z will cause different proportional variations of the computed deflections (equation 34) and stresses (equation 36 or 37). After some adjustments of the estimated values of Z and L it should be possible to establish a fair agreement between the computed and observed values of deflections and stresses and the value of E obtained by tests of cylinders or control beams.

EXTREME CASE IN WHICH THE SUBGRADE IS ASSUMED TO ACT AS A SOLID WITH CONSTANT MODULUS OF ELASTICITY

The preceding theory may be applied in the study of this special case. The problem is to determine L and Z so that the deflections of the subgrade due to the load, $kz+q$, will agree fairly closely with the deflections, z'', of the slab. Except for the possibility of a local heaving of the subgrade directly under the load, the value of Z determined in this way may be assumed to be the upper limit of Z. The case is of interest for this reason. The value $Z=0$ may be assumed to be a lower extreme, except for the possibility of a low or soft spot in the subgrade directly under the load.

An approximate analysis was made under the assumption that the subgrade has a constant modulus of elasticity in compression, E_s, and a constant Poisson's ratio, μ_s. This analysis gave the results.

$$L = 5l,\ Z = 0.3906,\ K = 0.1242\frac{E_s}{1-\mu_s^2} \quad \text{---} \quad (38)$$

By substituting these values of L and Z in equations 34 and 37 one obtains, for the particular case,

$$z''_i = 0.07617\frac{P}{D} \quad \text{---} \quad (39)$$

and

$$\sigma''_i = 1.1(1+\mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{h}{b}\right) + \frac{1}{3}\log_{10}\left(\frac{E}{K}\right) - 0.302\right] \quad \text{---} \quad (40)$$

The maximum reaction per unit of area was found to be $0.350\frac{P}{l^2}$ instead of $0.125\frac{P}{l^2}$ as obtained in the original theory.

Under actual conditions it must be expected that $0<Z<0.39$. Then equations 34 and 37 will apply instead of equations 39 and 40.

AN IMPROVED RECORDING STRAIN GAGE

Reported by L. W. TELLER, Senior Engineer of Tests, U.S. Bureau of Public Roads

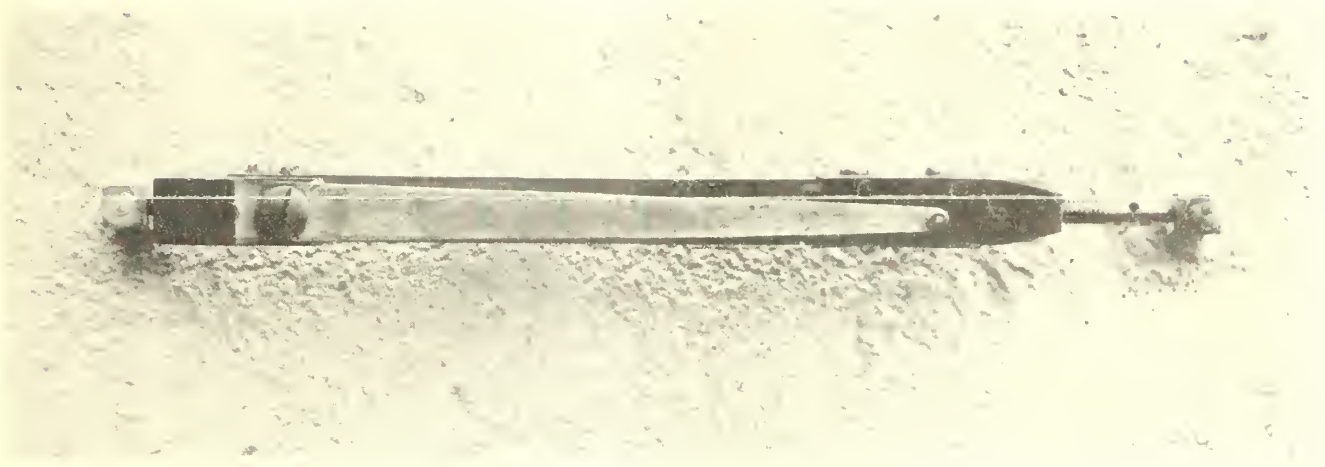

FIGURE 1.—EARLY TYPE OF RECORDING STRAIN GAGE.

SEVERAL years ago there was developed in the Division of Tests a small recording strain gage designed primarily for the determination of stresses in concrete structures. This gage, which has been described in detail elsewhere[1] was about 6 inches in length, of small cross section, employed a single bell-crank lever giving a multiplication ratio of about 60 to 1, and recorded on a smoked glass slide. It is shown in figure 1.

Subsequent to its development, extensive use was made of the instrument in a number of research projects dealing with stress conditions in concrete structures.[2]

Experience with the gage on the projects mentioned demonstrated the value of a strain gage of this size and type and indicated certain defects in the existing design, the elimination of which would improve both the quality of the data obtained and the mechanical functioning of the instrument. For example, the original gage was built with the gage body of brass, a metal poorly suited to the purpose because its high conductive and high expansive properties made the gage very sensitive to thermal changes. Mechanically there were difficulties with the manipulation of the slide on which the record was made and with the functioning of the fulcrum of the bell-crank lever. Fundamentally, the principle of the gage appeared to be satisfactory and with the idea of overcoming the temperature and mechanical troubles mentioned above the instrument was completely redesigned, retaining the principle of the original gage but introducing new materials and new mechanical features wherever it appeared that these would improve the functioning of the instrument.

Several gages were built according to the new design and these have now been in use for more than a year. Careful calibration tests and a very considerable amount of field work with these improved gages indicate that the objects sought in the redesign have been largely attained. Since a satisfactory gage of this type can be put to many uses, it was thought that a complete description would be of general interest.

TEMPERATURE EFFECTS IMPORTANT IN DESIGN OF STRAIN GAGE

Changes in temperature cause dimensional changes in most materials, and probably the most serious difficulty usually encountered in making strain measurements is the separation of the linear changes due to stress from those which are caused by thermal changes either in the material being stressed or in the gage itself. In the case of measurements in concrete, the thermal changes in the material take place rather slowly because of the low thermal conductivity of the concrete, and the dimensional changes in the material occur at a correspondingly low rate. Unless the stressing of the concrete is extended over a period of time sufficient to permit the concrete to change in temperature by about one degree centigrade or more the effect of temperature on the material does not ordinarily have to be taken into account. The rapidity with which the metal in the gage itself responds to thermal changes makes the effect of such changes on the dimensions of the gage an important consideration in the design, the ideal design being one in which complete compensation for the effect of changes in gage temperature is provided.

As previously noted the original gages were built with bodies of brass. This metal is highly conductive and has relatively a high coefficient of thermal expansion (about 0.000019 per degree centigrade). The result is that a gage built of brass would change temperature very quickly and the changes in the length of the gage would be relatively large. For example, if one assumes that the modulus of elasticity for concrete is 4,000,000 pounds per square inch it is apparent that the elongation of a brass gage due to a temperature rise of 1° C. would equal the elongation of the concrete due

[1] Pocket Strain Gage Gives Stresses in Concrete Roads, A. T. Goldbeck, Engineering News-Record, Mar. 29, 1923.

[2] Cf. the following publications:

1. Impact Tests on Concrete Pavement Slabs, by Leslie W. Teller, PUBLIC ROADS, vol. 5, no. 2, April 1924.
2. Static Load Tests on Pavement Slabs, by J. T. Thompson, PUBLIC ROADS, vol. 5, no. 9, November 1924.
3. Stress Measurements in Concrete Pavements, by L. W. Teller, Proc. 5th Annual Meeting, Highway Research Board, 1925.
4. The Six Wheel Truck and the Pavement, by L. W. Teller, PUBLIC ROADS, vol. 6, no. 8, October 1925.
5. Static and Impact Strains in Concrete, by J. T. Thompson, PUBLIC ROADS, vol. 7, no. 5, July 1926.
6. Tests of the Delaware River Bridge Floor Slabs, by Geo. W. Davis, PUBLIC ROADS, vol. 8, no. 8, October 1927.

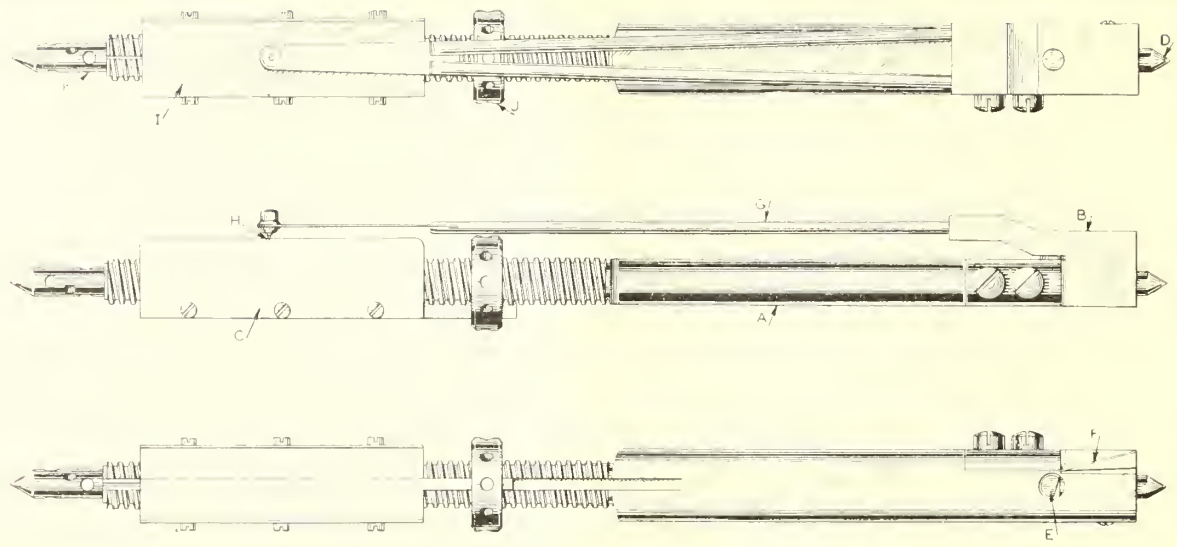

FIGURE 2.—IMPROVED TYPE OF RECORDING STRAIN GAGE.

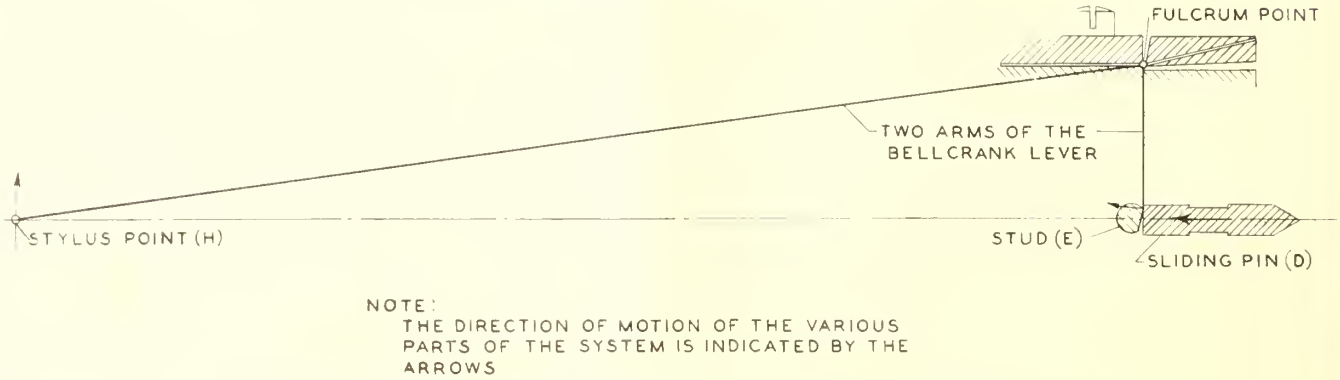
FULCRUM POINT
TWO ARMS OF THE BELLCRANK LEVER
STYLUS POINT (H)
STUD (E)
SLIDING PIN (D)
NOTE:
THE DIRECTION OF MOTION OF THE VARIOUS PARTS OF THE SYSTEM IS INDICATED BY THE ARROWS

FIGURE 3.—DIAGRAM SHOWING THE MECHANICAL FUNCTIONING OF THE GAGE.

to a tensile stress of 76 pounds per square inch. This is approximately one fifth to one fourth of the ultimate strength in tension of the concrete; and when one considers that variations of several degrees in the temperature of the air surrounding the strain gage are not uncommon, the seriousness of this temperature effect is evident.

DESCRIPTION OF IMPROVED GAGE

Three views of the redesigned instrument are shown in figure 2. Figure 3 is a diagram illustrating the manner in which the gage functions, and figure 4 is a "phantom" sketch showing in perspective the relative positions of the essential parts of the instrument.

The gage consists of three major parts—a gage body (A) on which is mounted a bell-crank lever (B) and a slide carriage (C). In one end of the gage body there is a freely sliding pin (D), the outer end of which is conical and rests in one of the gage points, while the inner end is a plane surface which bears against the knife edge of the stud or post (E). The latter is fastened rigidly to the bell-crank lever and moves freely in the oversize hole in the gage body. The distance from this knife edge to the fulcrum plate or spring hinge (F) constitutes the short arm of the bell-crank lever, while the long arm is formed by the two metal rods (G) carrying the stylus point (H). This stylus point bears against the smoked surface of a small glass plate (I) mounted in the slide carriage. After each record is made this carriage is moved along the body of the gage by rotating the capstan nut (J) on the threaded portion of the gage body. An adjustable conical point (K) in the end of the gage body nearest to the slide carriage rests in the other gage point.

OPERATION OF GAGE DESCRIBED

In use the gage is placed between the gage points as shown in figure 5 and the length is adjusted until the ends of the gage rest firmly in the gage points and the stylus point is in a position about half-way across the glass slide. In this position either tensile or compressive deformations can be measured.

When a change in the distance between the gage point occurs, the sliding pin D moves longitudinally in the gage body and this translation causes a rotation of the stud (E) about the fulcrum. This stud being a part of the bell-crank lever, a corresponding motion of the stylus point across the glass slide occurs, the magnitude of the two displacements being in the ratio of the lengths of the respective arms of the bell crank lever, or about 60 to 1. This movement of the stylus point produces a trace in the smoke film on the glass and the length of this trace obviously bears a direct relation to the change in distance between the gage points. After the recording of the deformation has been completed the capstan nut (J) is rotated, moving the slide carriage longitudinally to a new position.

Twenty or more records can be made on one glass slide. The appearance of two typical completed records on the glass slide is shown in figure 6.

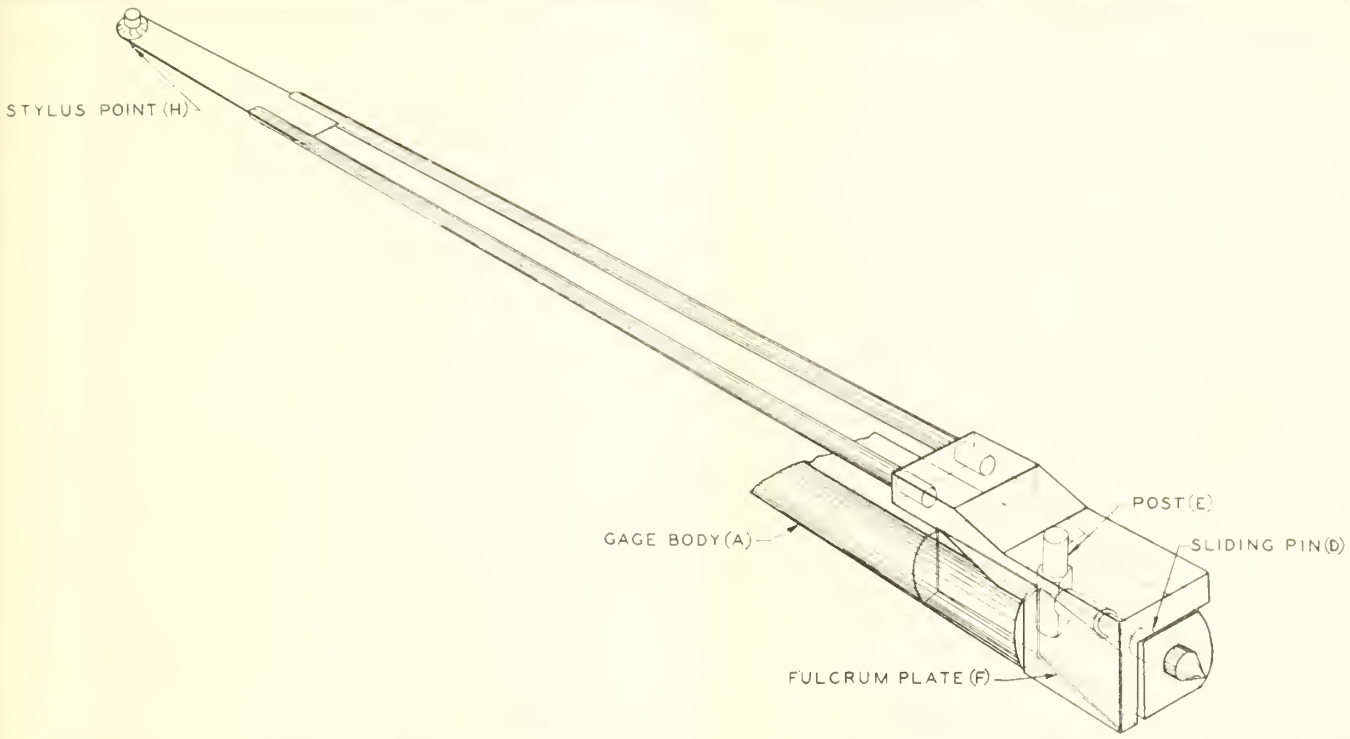

FIGURE 4.—PHANTOM SKETCH, SHOWING RELATIVE POSITIONS OF WORKING PARTS OF GAGE.

FIGURE 5.—IMPROVED TYPE OF RECORDING STRAIN GAGE MOUNTED BETWEEN GAGE POINTS. VIEW LOOKING DOWN ON TOP OF INSTRUMENT.

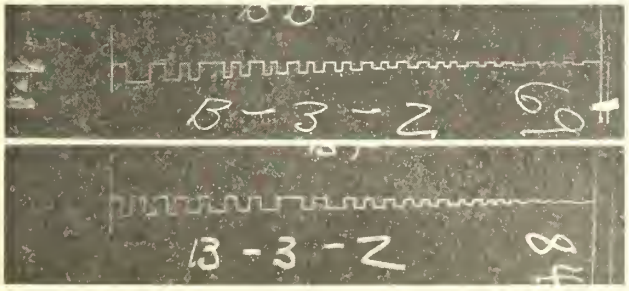

FIGURE 6.—COMPLETED STRAIN-GAGE RECORDS. THESE SLIDES HAVE BEEN FIXED AND ARE READY FOR MEASUREMENT OF THE LENGTH OF THE DISPLACEMENTS IN THE DIRECTION PERPENDICULAR TO THE LONG AXIS OF THE SLIDE.

COMPENSATION FOR THERMAL EXPANSION DEVISED

In designing the gage it was decided to reduce the thermal expansion to a minimum through the use of the steel-nickel alloy known as "Invar." This material has a guaranteed coefficient of thermal expansion of not to exceed 0.8×10^{-6} (or about 4 percent of that of brass). The entire gage body from one gage point to the other is made of this metal. But even this material changes in dimension when subjected to temperature changes; and in order to compensate within the gage for such changes the stylus arm was designed with two metal rods having slightly different coefficients of thermal expansion and so spaced as to curve the stylus arm to an extent sufficient to compensate for the rotation of the bell crank caused by the expansion or contraction of the gage parts. While it would probably have been possible to compute the length and spacing of these rods, it was found better in this instance to adjust these dimensions by trial.

To do this the frame shown in figure 7 was built. A steel having a known coefficient of thermal expansion was used for the purpose. As will be noted in the photograph the side members are hollow and contain two mercurial thermometers. The gage was mounted between the cross members of the frame. A wooden box which provided a support for the frame and served to maintain a uniform temperature around it completed the apparatus.

With the strain gage in the frame, the whole was subjected to changes in temperature greater than would be encountered in the normal operation of the gage and from the data obtained the error due to gage

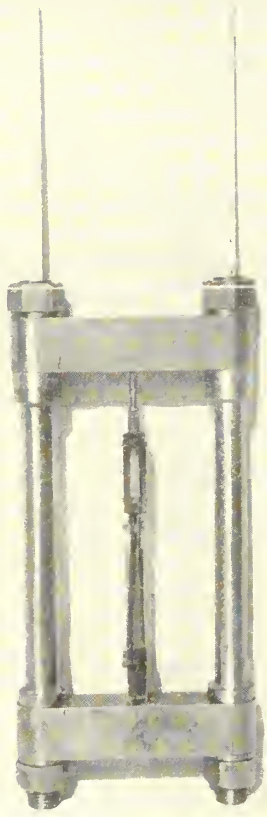

FIGURE 7.—STEEL FRAME USED FOR DETERMINING THE EFFECT OF CHANGES IN THE TEMPERATURE OF THE STRAIN GAGE AND FOR ADJUSTING THE BIMETALLIC ARM.

expansion was determined. The compensating feature of the stylus arm was adjusted, by trial, until the length of the trace in the smoke film on the strain-gage slide was that corresponding to a change in gage length equal to the expansion (or contraction) in the steel frame.

While perfection was not attained in this adjustment the compensation feature has reduced the error arising from temperature changes in the gage to a point where it is negligible for the usual operating conditions under which the gage is used.

OTHER IMPROVEMENTS MADE

As will be noted by reference to figures 2, 3, and 4 the bell-crank lever operates about a plate-fulcrum or spring hinge. The substitution of steel for brass in the supporting parts for this steel spring hinge apparently has eliminated a trouble which was frequently had with the original design, caused by the separation of the spring from the supporting metal near the fulcrum point. Such separation resulted in a slight shifting of the flexure point of the spring and a corresponding change in the magnification ratio of the gage.

The slide carriage used on this gage is new. The glass slide is mounted between bronze spring clips which are fastened to the sides of the slide carriage. The carriage itself slides over the threaded portion of the gage body, being kept from rotating by a spline which slides in a keyway in the body of the gage. This is shown clearly in figure 2. A capstan nut threaded on the gage body operates in a rectangular notch in the extended end of this spline. Rotation of the nut moves it along the gage and draws with it the spline and the slide carriage to which the spline is attached.

CALIBRATIONS MADE ON SPECIAL MACHINE

For the calibration of these gages and similar linear measuring instruments, the special calibrating device shown in figure 8 was designed and built.

It consists of a cast-iron base carrying three pedestals. In the pedestal at the right (see fig. 8) there is a stationary cylindrical steel block which may be adjusted horizontally to any desired position and clamped. In the pedestal at the left is a long nut into which is threaded a precise micrometer screw. This screw is provided with a graduated drum reading directly to 0.0001 inch and a knurled operating knob at its outer end. Its inner end is finished with a plane surface of hardened steel. The combination of screw, nut, and graduated drum was calibrated for accuracy of displacement indication by the National Bureau of Standards and the errors determined to the nearest one hundred thousandth of an inch.

In the center pedestal is a movable steel block of triangular cross section which slides longitudinally in V bearings under the action of the micrometer screw. The movable block is fitted with a spherical surface where it makes contact with the end of the screw. This block is kept seated in its V guides by spring-pressed rollers which bear on its upper surface. A firm contact between the block and the micrometer screw is maintained by means of a suspended weight which exerts a longitudinal pull on the movable block through a flexible ribbon operating over a pulley.

In use the gage to be calibrated is mounted between the fixed block and the movable block as shown in figure 8. The movable block is then displaced horizontally definite amounts by means of the micrometer screw, the amount of the displacement being determined by the graduations on the drum. After each displacement the glass record slide on the strain gage is moved ahead to a new position. It has been customary to use displacement increments of five ten-thousandths of an inch, to repeat each setting several times, and to have each gage calibrated independently by two operators.

The glass slide on which the record is made is coated with a light smoke film. After the record is complete and such identifying numbers as are desired have been marked in the film, the record is fixed by a coating of copal gum dissolved in acetone. These records (see fig. 6) may be measured satisfactorily in either one of two ways. The length of the trace scribed by the stylus point may be measured directly with a measuring microscope or a linear comparator, or the glass slide may be mounted in a projection apparatus and an enlarged image thrown on a screen containing rectangular coordinates by means of which the length of the projected trace is measured. While both methods have been used in the work of the Bureau of Public Roads, the latter method has been found more convenient. For this work a projection apparatus was built, the magnifying device being an ordinary laboratory microscope. This projector gives a magnification of approximately 30:1, which, combined with the gage magnification of about 60:1, gives a total magnification of about eighteen hundred times the original displacement.

Figure 9 is the calibration curve of one of these gages, the true displacement and the length of the

FIGURE 8. SPECIAL CALIBRATING DEVICE FOR THE APPLICATION AND PRECISE MEASUREMENT OF SMALL LINEAR DISPLACEMENTS TO STRAIN GAGES, MICROMETER DIALS, AND SIMILAR APPARATUS.

projected record on the strain gage slide being used as the coordinates. The multiplication ratio is constant and for this particular gage is shown to be slightly less than 58:1.

PERFORMANCE CHARACTERISTICS ARE SATISFACTORY

The accuracy of strain determination possible with this gage depends principally on the accuracy with which the travel of the stylus point in the smoke film can be measured. This in turn depends upon the sharpness of the record left by the stylus. The smoke film is a layer of carbon which is scraped aside by the stylus as it moves across the slide. Some of the carbon piles up ahead of the stylus and remains in a minute heap at the end of the trace. The blunter the stylus point and the thicker the smoke film the broader will be the record, the larger will be this heap, and the more difficult will be the determination of the true distance traveled by the point. Hence, the accuracy of the measurement can be considerably increased if the stylus point is kept sharp and if the smoke film on the glass slide is made as thin as possible.

If these two points are given attention the record will be clearly defined and its length, when projected on the screen, can be measured with certainty to the nearest 0.05 inch. Then, since the gage length is about 6 inches and the combined magnification of the strain gage lever and of the projection apparatus is approximately 1800:1, the least unit deformation which could be measured definitely would be $\frac{0.05}{1800 \times 6}$ or 0.0000046 inch per inch.

In concrete, if we assume a modulus of elasticity of 4,000,000 pounds per square inch, this deformation corresponds to a unit stress of 18 pounds per square inch. In the work which the Bureau has done with these gages it has been considered that the stress determinations were probably accurate to 20 or 25 pounds per square inch.

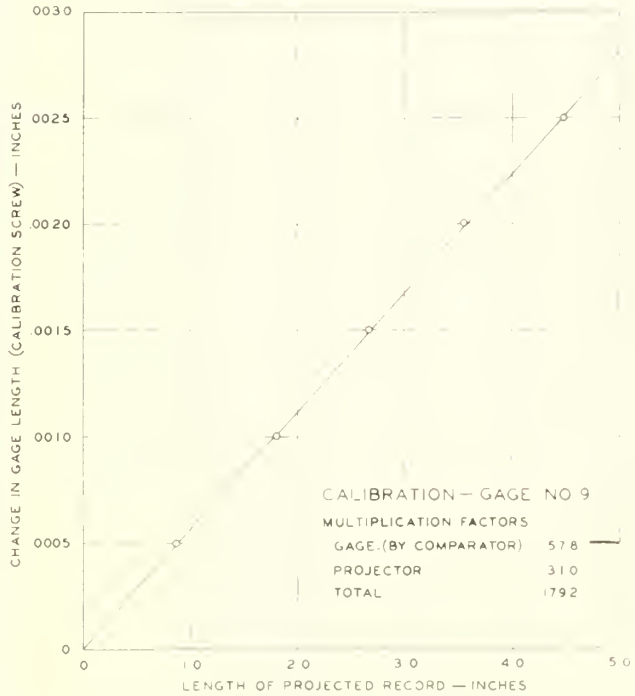

FIGURE 9. CALIBRATION CURVE OF A RECORDING STRAIN GAGE OF THE IMPROVED TYPE.

The consistency of performance can best be illustrated by the following table, which shows the measured length of projected record for 24 repetitions of a total displacement of 0.0010 inch.

The average of the 24 measured lengths is 2.015 inches. The maximum deviation is 5.7 percent, the

average deviation is 1.6 percent, and only 3 observations (tests nos. 3, 15, and 21) deviate more than 2 percent from the mean of the group of 24.

Up to the present time the strain gages have been used entirely for the measurement of strains induced in concrete pavement slabs by slowly applied loads and no study has been made of their behavior when subjected to the sudden deformations caused by impact loads. It is planned to make such a study later. Under the conditions noted, many thousands of measurements have been made and the behavior of the gages throughout the work has been highly satisfactory.

TABLE 1.—*Calibration data for gage no. 8; true displacement 0.0010 inch*

Test no.	Length of record	Test no.	Length of record	Test no.	Length of record
	Inches		*Inches*		*Inches*
1	2.000	9	2.000	17	2.000
2	2.050	10	2.025	18	2.050
3	1.950	11	2.025	19	2.050
4	2.000	12	2.025	20	2.000
5	2.050	13	2.050	21	1.925
6	2.050	14	2.050	22	2.000
7	2.050	15	1.900	23	2.050
8	2.050	16	2.000	24	2.000

(Continued from page 184)

SUMMARY

In conclusion, attention is called to certain practical considerations of the problem as regards (*a*) the moisture content required to furnish the desired fluidity and (*b*) the definition of detrimental shrinkage. From the foregoing discussion it may be inferred that the desired consistency of mix is furnished by only one moisture content which must be determined with a high degree of exactness. It seems likely that pastes having the low consistency required in mud-jack work may be equally suitable with moisture contents varying within a range of 5 percent or more.

A water content of 42 percent has been suggested for the mixture using soil S-6361 but in practice the consistency of mix is determined by the operator on the basis of visual inspection. Successive batches probably had moisture contents varying from about 40 to 45 or 46 percent and all appeared to work equally well. An experienced laboratory worker, on the basis of visual inspection, would not be able to determine more accurately the moisture required to furnish equal consistency in several mixtures made from the same soil.

If the flow curve can be used to indicate the required moisture content within possibly 5 percent, its use for the purpose proposed is warranted. Whether the optimum water content for all soils is that corresponding to 0.1 blow, as was the case for soil S-6361 must be determined by additional investigation.

It is known that detrimental shrinkage cannot occur when the shrinkage limit is equal to or greater than the moisture content of the mixture. It is not known how much the shrinkage limit can be below this moisture content and still avoid harmful shrinkage.

A 7-percent portland cement admixture completely eliminated the possibility of shrinkage of paste made with soil S-6361 and this is desirable whenever it can be accomplished. As a matter of fact, had this paste contained as much as 49 percent of moisture it would not have shrunk on loss of moisture since the shrinkage limit of the mixture with 7 percent of cement was 49. However, soils of the excellent character of S-6361 are not available in many parts of the United States. In some cases it may be impractical to add enough cement to the soils which must be used, to completely eliminate all possibility of shrinkage. However, a conception of the basic physical requirements of pastes for use in mud jacking and a knowledge of the simple laboratory tests for disclosing the properties of soils and soil mixtures will assist materially in the selection of the best soils at hand.

A laboratory investigation of this character should include the following steps:

1. Hydrometer method of analysis to determine the grading.

2. Determination of the flow curve of the soil without admixture to furnish the liquid limit and give some idea of the moisture content at the desired consistency of mix.

3. Determination of the shrinkage limit corresponding to several admixtures of cement to determine the effect of cement in reducing the shrinkage of the mix on loss of moisture.

For all practical purposes it can be assumed that the mixture with the cement admixture will require several percent more moisture than that of the soil and water without the admixture.

The possibilities of soil admixtures for improving the grading of soils should not be overlooked. Other materials which have been suggested as admixtures in soil stabilization [3] may be found useful.

It may be found impractical to eliminate all shrinkage by admixture of portland cement and possibly fine sand but the tests will show which mixtures can be most easily improved by such means and consequently which soils of a group are apt to make the best mixtures.

It is safe to assume that all soils which the suggested laboratory tests show to have the same characteristics as soil S-6361 are physically the same and therefore should duplicate the good performance of this soil in mud jacking. We are not in a position, however, to say how far the characteristics of soils may vary from those of S-6361 and still be suitable for mud jacking.

The four soils described in this report illustrate the great variation which may exist in the character of soils selected for mud jacking by visual inspection on the basis of our present knowledge of the subject. They emphasize the need for additional information regarding the character of such soils and tests for properly identifying them.

[3] See subgrade soil studies, p. 38.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS III—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF NOVEMBER 30, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	Public works funds	Mileage	Estimated total cost	Public works funds allotted	Percentage completed	Mileage	Public works funds allotted	Mileage		
	$	$	$		$	$			$		$	
Alabama	2,092,533								85,000.00	13.5	2,007,533.00	Alabama
Arizona	625,435				40,700.00	40,700.00		6.0			584,735.00	Arizona
Arkansas	1,687,084								88,098.85	1.7	1,598,985.15	Arkansas
California	3,901,838				637,625.93	499,573.95	7.2	26.0	763,429.57	73.0	2,638,834.48	California
Colorado	1,718,632				689,262.42	689,262.42	15.5	139.9	185,000.00	41.9	844,369.58	Colorado
Connecticut	659,120				211,882.58	211,882.58		3.1			447,237.42	Connecticut
Delaware	454,772										454,772.00	Delaware
Florida	1,307,958				500,816.33	500,816.33	3.0	25.6	576,073.59	42.8	231,068.08	Florida
Georgia	2,320,973								76,055.21	6.8	2,244,917.79	Georgia
Idaho	1,121,562	11,625.34	7,523.08	2.8	540,602.96	540,602.96	30.0	103.1	94,358.09	10.5	479,077.87	Idaho
Illinois	6,262,223				250,311.61	250,311.61	4.0	29.9	1,536,124.68	128.7	4,475,786.71	Illinois
Indiana	501,892				77,000.00	77,000.00		19.1	88,000.00	29.0	336,892.00	Indiana
Iowa	2,212,245	28,117.41	26,600.00	4.6	532,025.44	507,300.00	19.3	37.8	164,600.00	14.4	1,513,745.00	Iowa
Kansas	2,522,401				10,651.49	10,651.49		3.4	826,662.38	62.8	1,685,087.13	Kansas
Kentucky	1,879,340				403,334.21	403,334.21	.5	33.3	1,360,915.80	180.1	115,089.99	Kentucky
Louisiana	1,457,148				59,390.34	59,390.34	40.4	2.3	365,389.88	14.4	1,032,367.78	Louisiana
Maine	842,479	186,151.34	186,151.34	23.2	673,972.74	629,407.73	87.2	71.7	26,634.85	.2	285.08	Maine
Maryland	891,132								115,754.25	7.7	775,377.75	Maryland
Massachusetts	527,768				367,127.97	367,127.97	39.0	11.5			160,640.03	Massachusetts
Michigan	3,184,057				553,250.00	553,250.00	.5	24.0	1,236,350.00	120.1	1,394,457.00	Michigan
Minnesota	2,131,314				904,177.04	896,429.91	17.8	161.9	604,936.14	95.7	629,947.95	Minnesota
Mississippi	1,744,669				435,000.00	435,000.00	10.8	54.6			1,309,669.00	Mississippi
Missouri	3,045,076				702,288.32	702,288.32	7.5	134.2	754,102.18	132.8	1,588,685.50	Missouri
Montana	1,859,937				120,587.26	120,587.26	2.5	4.4	355,941.47	50.4	1,383,408.27	Montana
Nebraska	1,957,240				1,101,827.06	1,099,964.74	5.5	238.9	126,850.25	21.8	730,425.01	Nebraska
Nevada	1,136,479				358,863.14	338,863.14	13.6	52.4	201,083.14	14.9	596,532.72	Nevada
New Hampshire	477,460				234,358.45	234,358.45	29.0	9.7	99,882.19	6.7	143,219.36	New Hampshire
New Jersey	63,460								56,550.52	.5	6,909.48	New Jersey
New Mexico	1,448,234				522,500.00	522,500.00	1.7	250.8	256,000.00	225.0	669,734.00	New Mexico
New York	3,662,137	6,300.00	6,300.00	.1	3,082,363.19	2,884,063.19	15.1	73.2	767,300.00	19.1	4,473.81	New York
North Carolina	2,380,573				507,308.33	502,026.56	16.3	40.0	364,501.77	30.5	1,514,044.67	North Carolina
North Dakota	1,451,112								46,035.01	10.4	1,405,076.99	North Dakota
Ohio	3,871,148				793,500.00	688,064.90	13.1	180.4	2,213,230.00	75.7	969,853.10	Ohio
Oklahoma	2,304,199								41,159.13	.7	2,263,039.87	Oklahoma
Oregon	1,526,724				168,868.15	168,868.15	1.2	7.8	532,412.39	60.6	825,443.46	Oregon
Pennsylvania	7,716,975				4,500,948.76	4,500,948.76	11.4	478.4	2,131,374.16	170.8	1,084,652.08	Pennsylvania
Rhode Island	499,677								9,889.00	1.1	489,788.00	Rhode Island
South Carolina	1,364,791				766,612.29	766,612.29	2.1	96.9	204,216.54	25.0	393,962.17	South Carolina
South Dakota	1,502,870				80,636.49	80,636.49	38.4	30.9			1,422,233.51	South Dakota
Tennessee	2,123,155				87,972.50	87,972.50		8.5	463,303.83	52.5	1,571,878.67	Tennessee
Texas	6,061,006				1,472,019.59	1,106,072.58	9.9	210.4	1,483,928.00	308.9	3,471,005.42	Texas
Utah	1,048,677	61,543.36	61,543.36	8.0	457,339.35	457,339.35	2.6	122.3	191,342.66	8.9	338,451.63	Utah
Vermont	465,026				241,426.64	232,346.12	6.0	21.3	94,931.88	8.9	137,748.00	Vermont
Virginia	1,854,189				815,399.51	815,399.51	17.4	100.2	246,298.59	22.8	792,490.90	Virginia
Washington	1,180,362				449,857.95	449,857.35	15.8	34.6	203,576.55	10.6	526,928.10	Washington
West Virginia	1,118,559				83,064.57	83,064.57	3.6	2.8	147,135.19	9.4	888,359.24	West Virginia
Wisconsin	2,431,220				1,180,135.41	1,141,120.13	19.5	83.2	97,138.48	.5	1,192,961.39	Wisconsin
Wyoming	1,125,332				461,759.26	461,759.26	6.1	79.8	113,966.64	13.5	549,606.10	Wyoming
District of Columbia	767,388				330,901.12	330,901.12	54.1	3.4	385,739.86	2.1	50,747.02	District of Columbia
Hawaii	187,106										187,106.00	Hawaii
TOTALS	94,676,687	293,737.45	288,117.78	38.7	25,407,668.40	24,447,656.24	14.5	3,017.7	19,781,272.72	2,127.4	50,159,640.26	TOTALS

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

SUMMARY OF CLASSES I, II, AND III

AS OF NOVEMBER 30, 1933

STATE	TOTAL APPORTIONMENT OF PUBLIC WORKS FUNDS	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR ALL NEW PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	8,370,133					2,996,456.19	1,660,289.19	1,334,167.00	8.4	142.7	1,092,600.55	123.7	5,617,243.26
Arizona	5,211,960					2,447,665.92	2,447,665.92		15.2	174.6	309,079.55	21.9	2,455,214.53
Arkansas	6,748,335					989,890.89	667,420.55	322,470.34	12.4	23.8	666,488.77	44.4	5,414,425.68
California	15,607,354					6,769,411.61	5,658,509.28		9.6	212.4	2,454,347.34	134.1	7,494,497.38
Colorado	6,874,530	132,258.17	132,258.17		5.1	2,188,158.30	2,188,158.30		31.4	216.7	934,551.60	67.9	3,619,561.93
Connecticut	2,865,740					1,056,975.87	1,056,975.87		13.5	11.3	1,091,170.93	19.5	717,593.20
Delaware	1,819,088					673,933.20	673,933.20		37.5	24.4	456,536.30	12.7	688,618.50
Florida	5,231,834					2,963,558.27	2,041,004.84	921,943.33	10.2	108.5	885,944.42	50.5	2,304,884.74
Georgia	10,091,185					363,200.66	363,200.66		14.6	14.7	1,224,292.80	58.4	8,503,691.54
Idaho	4,486,249	64,589.98	59,357.80		22.3	2,305,857.81	2,290,924.76		34.2	219.5	334,360.09	18.5	1,801,606.35
Illinois	17,570,770					1,362,138.02	1,362,138.02		3.9	47.8	5,613,110.79	172.1	10,595,521.19
Indiana	10,037,843					1,988,204.83	1,988,204.83		17.2	86.0	1,199,981.89	59.5	6,849,656.28
Iowa	10,055,660	821,647.87	769,150.00		33.7	5,092,476.46	4,801,800.00		25.1	236.5	1,345,030.00	84.5	3,139,680.00
Kansas	10,089,604					1,916,589.18	1,916,589.18		19.0	200.0	3,911,541.24	279.3	4,261,473.58
Kentucky	7,517,359					1,897,739.20	1,897,739.20		10.5	146.7	2,510,592.63	247.0	3,109,027.17
Louisiana	5,828,591					873,847.98	873,847.98		39.1	27.3	1,703,657.67	44.1	3,251,085.35
Maine	3,369,917	226,018.23	226,018.23		24.0	2,028,003.30	1,978,027.18		51.6	110.7	508,108.81	14.3	657,762.78
Maryland	3,564,527					173,094.69	159,295.94		15.7	5.4	790,368.20	17.7	2,614,862.86
Massachusetts	6,597,100					3,075,711.12	2,656,444.21	412,620.05	15.4	52.0	273,771.01	6.1	3,666,884.78
Michigan	12,736,227	17,500.00	17,500.00		.8	2,320,820.00	2,313,670.00		33.8	117.5	3,754,701.00	189.4	6,645,356.00
Minnesota	10,656,569	1,105,724.69	1,105,724.69		161.9	3,242,997.63	3,235,250.48		32.7	469.7	1,474,064.52	179.1	4,841,529.31
Mississippi	6,978,675					1,162,776.36	801,570.63	322,573.79	13.6	91.4	312,984.42	30.9	5,864,119.95
Missouri	12,180,306					3,907,101.05	3,583,043.07		25.7	234.2	1,979,657.16	173.2	6,617,605.77
Montana	7,439,748	16,001.35	16,001.35		.5	3,575,421.30	3,395,421.30	100,000.00	21.1	273.9	1,056,141.73	102.6	2,972,183.62
Nebraska	7,828,961	143,918.11	125,832.56		45.8	5,120,648.07	4,334,969.61	108,538.00	29.9	496.4	976,701.91	86.0	2,391,456.92
Nevada	4,545,917	24,027.31	24,027.31		10.0	1,544,956.41	1,524,956.41		19.5	204.9	846,182.27	40.3	2,150,751.01
New Hampshire	1,909,839					869,951.82	869,951.82		22.9	21.5	175,009.86	8.2	864,877.32
New Jersey	6,346,039					1,387,179.29	1,387,179.29		25.4	13.4	1,931,310.10	16.0	3,027,549.61
New Mexico	5,792,935	164,325.27	164,325.27		26.7	2,893,456.85	2,893,456.85		17.5	444.6	439,504.91	246.9	2,295,647.97
New York	22,330,101	26,300.00	26,300.00		.5	15,423,937.99	14,879,563.19	300,000.00	15.2	286.4	6,021,850.00	81.7	1,402,387.81
North Carolina	9,522,293	118,146.93	67,958.93	50,188.00	17.1	1,935,399.23	1,411,454.77	518,662.69	22.3	189.5	1,233,884.64	158.5	6,808,994.66
North Dakota	5,804,448	217,402.66	217,402.66		146.8	1,041,764.37	1,041,764.37		26.0	304.8	1,329,013.83	316.9	3,216,267.14
Ohio	15,484,592	4,090.00	4,090.00		.1	6,580,290.00	6,384,879.90		27.1	316.4	4,732,155.00	134.7	4,363,467.10
Oklahoma	9,216,798					2,884,618.52	2,884,618.52		15.8	170.6	1,346,675.15	74.5	4,985,504.33
Oregon	6,106,896	137,000.56	123,632.06		30.9	2,680,955.63	2,650,534.63		16.1	115.9	1,186,430.88	101.7	2,146,298.43
Pennsylvania	18,891,004					6,750,623.31	6,667,531.22		10.2	538.7	6,582,624.74	264.4	5,640,848.04
Rhode Island	1,998,708					947,533.05	947,533.05		23.2	19.8	247,517.15	4.6	803,657.80
South Carolina	5,459,165					1,905,174.17	1,902,039.54	3,134.63	12.3	224.3	625,665.75	71.5	2,931,459.71
South Dakota	6,011,479	558,953.88	558,953.88		63.8	1,523,708.87	1,488,826.75	34,882.12	10.8	246.1	479,354.96	25.0	3,484,343.41
Tennessee	8,492,619					1,339,874.27	915,516.55	424,357.72	22.7	46.3	1,225,903.87	91.8	6,351,198.58
Texas	24,244,024					6,766,934.03	5,374,106.21		12.3	710.1	5,018,433.53	605.0	13,851,484.26
Utah	4,194,708	421,042.89	421,042.89		89.4	2,286,455.85	2,285,049.77		48.7	221.2	477,953.67	15.3	1,010,661.67
Vermont	1,867,573	41,694.23	41,694.23		5.8	1,140,388.72	1,130,289.59		7.3	60.0	260,865.87	14.1	434,723.31
Virginia	7,416,757	37,687.80	26,918.09	10,769.71	2.9	2,037,649.30	2,022,475.65	15,173.65	20.8	146.9	1,774,961.56	60.1	3,592,401.70
Washington	6,115,867					2,312,445.28	2,309,087.86	3,356.82	23.9	106.6	838,415.86	29.2	2,968,363.28
West Virginia	4,474,234					1,773,351.47	1,773,351.47		30.3	72.7	428,793.25	15.3	2,272,089.28
Wisconsin	9,724,881	319,550.73	319,550.73		12.1	4,333,363.17	4,271,219.50	21,500.00	31.2	205.6	713,823.31	34.1	4,420,287.46
Wyoming	4,501,327	234,014.99	222,400.00		64.0	2,211,969.33	2,076,009.43	60,000.00	18.4	386.9	553,603.20	83.1	1,649,314.37
District of Columbia	1,918,469	113,402.25	113,402.25		.8	1,003,853.38	1,003,853.38		29.4	5.9	588,429.20	3.4	212,784.17
Hawaii	1,871,062					422,882.57	205,708.64	167,174.18	2.9	10.1	700,841.31	14.2	964,512.05
TOTALS	394,000,000	4,945,297.90	4,783,541.10	60,957.71	765.0	134,491,394.79	124,652,052.56	5,070,554.32	20.7	8,813.3	76,618,989.20	4,747.9	187,945,417.14

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS I—PROJECTS ON THE FEDERAL-AID HIGHWAY SYSTEM
OUTSIDE OF MUNICIPALITIES
AS OF NOVEMBER 30, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
Alabama	$ 4,185,067	$	$	$		$ 2,844,232.85	$ 1,565,417.08	$ 1,278,815.77	8.4	138.5	$ 929,901.95	106.5	$ 1,689,747.97
Arizona	3,804,731					2,395,517.67	2,395,517.67		15.3	167.6	309,079.55	21.9	1,100,133.78
Arkansas	3,374,167					814,858.62	524,850.81	290,007.81	14.7	19.4	462,605.75	38.8	2,386,710.44
California	7,803,677					5,696,894.23	4,806,534.54		9.7	182.2	1,130,156.18	48.1	1,866,986.28
Colorado	3,437,265	40,085.87	40,085.87		3.1	1,384,038.83	1,384,038.83		39.2	73.4	455,245.89	21.6	1,557,894.41
Connecticut	1,404,213					263,006.27	263,006.27		9.1	3.0	916,502.83	17.8	224,703.90
Delaware	909,544					474,569.30	474,569.30		41.7	22.4	422,915.30	12.1	6,059.40
Florida	2,615,917					1,998,510.81	1,248,506.15	749,392.53	13.9	77.4	293,266.31	7.3	1,074,142.51
Georgia	5,045,592					360,783.49	360,783.49		14.1	14.6	873,903.50	46.1	3,810,905.01
Idaho	2,243,125	51,214.29	50,084.37		18.9	1,562,746.81	1,549,143.66		38.6	111.3	117,770.95	6.3	526,126.02
Illinois	4,431,348					258,256.22	258,256.22		1.2	9.2	598,062.92	10.9	3,575,028.86
Indiana	4,717,786					1,886,760.23	1,886,760.23		18.1	66.5	1,111,981.89	30.5	1,719,043.88
Iowa	5,027,830	680,483.09	637,300.00		25.0	3,828,180.03	3,606,300.00		24.7	182.5	784,230.00	64.3	
Kansas	5,044,802					1,878,829.89	1,878,829.89		18.7	195.0	2,690,796.04	207.8	475,176.07
Kentucky	3,608,332					1,473,518.99	1,473,518.99		13.4	112.8	1,020,485.18	64.2	1,114,327.83
Louisiana	2,914,295					507,545.09	507,545.09		45.1	19.2	1,222,528.25	24.2	1,184,221.66
Maine	1,684,959					955,605.20	950,194.09		37.0	29.6	392,832.16	12.6	341,932.75
Maryland	1,782,263					156,305.93	146,657.18		13.6	4.8	674,613.95	10.0	960,991.87
Massachusetts	1,932,950					1,156,499.74	783,479.69	372,620.05	17.2	31.9	233,984.84	5.8	915,485.47
Michigan	5,094,491	17,500.00	17,500.00		.8	1,321,500.00	1,321,500.00		43.5	84.9	1,244,700.00	54.9	2,510,791.00
Minnesota	5,115,153	793,990.53	793,990.53		142.3	1,573,502.19	1,573,502.17		36.0	272.5	553,752.70	72.8	2,193,907.60
Mississippi	3,489,337					625,918.85	310,273.82	277,013.09	17.1	34.4	248,310.91	26.9	2,930,752.27
Missouri	6,090,153					2,581,149.77	2,265,581.29		32.3	93.6	1,062,847.58	38.0	2,761,724.13
Montana	4,463,849	16,001.35	16,001.35		.5	3,386,102.80	3,206,102.80	100,000.00	21.7	266.4	418,026.17	41.1	823,718.68
Nebraska	3,914,481	133,728.85	116,346.70		43.6	3,420,858.37	2,637,042.23	108,538.00	38.1	242.7	727,449.01	60.0	433,643.06
Nevada	2,909,387	24,027.31	24,027.31		10.0	1,135,789.00	1,135,789.00		18.1	151.3	545,099.13	25.4	1,104,471.56
New Hampshire	954,919					499,465.91	499,465.91		22.0	8.4	19,205.50	.4	436,247.59
New Jersey	3,065,137					327,297.82	327,297.82		18.9	6.1	867,650.43	8.4	1,870,188.75
New Mexico	2,896,467	164,325.27	164,325.27		26.7	2,207,062.77	2,207,062.77		22.2	189.6	113,226.89	20.7	411,852.07
New York	10,830,099					8,251,253.00	7,935,820.00	300,000.00	18.2	181.5	2,875,750.00	44.1	18,529.00
North Carolina	4,761,147	100,376.02	50,188.02	50,188.00	15.2	1,345,753.42	827,090.73	518,662.69	23.7	143.2	849,091.91	123.5	3,034,776.34
North Dakota	2,902,224	216,605.94	216,605.94		146.2	993,458.43	993,458.43		26.0	298.3	1,135,198.32	300.1	556,961.31
Ohio	6,968,066					4,777,440.00	4,777,440.00		30.2	123.4	1,882,140.00	49.6	308,486.00
Oklahoma	4,608,399					2,856,759.93	2,856,759.93		15.2	168.5	885,163.65	63.9	866,475.42
Oregon	3,053,448	137,000.56	123,632.06		30.9	2,062,136.40	2,031,715.40		18.9	101.3	555,402.94	36.7	342,697.60
Pennsylvania	5,757,978					1,757,596.82	1,757,596.82		6.0	49.6	2,833,797.34	64.2	1,166,583.84
Rhode Island	999,354					947,533.05	947,533.05		23.2	19.8	42,490.08	.7	9,330.87
South Carolina	2,729,583					1,073,432.02	1,070,702.40	2,729.62	18.7	121.1	364,514.74	40.7	1,294,365.86
South Dakota	3,005,739	536,458.58	536,458.58		62.6	1,238,135.27	1,203,253.15	34,882.12	7.0	205.2	374,578.92	21.4	891,448.35
Tennessee	4,246,309					1,218,124.60	793,766.88	424,357.72	23.7	36.2	755,633.88	38.8	2,696,908.24
Texas	12,122,012					4,893,525.30	3,893,029.42		14.5	475.8	2,459,686.41	243.7	5,769,316.17
Utah	2,097,354	332,342.68	332,342.68		81.0	1,337,798.65	1,337,798.65		58.5	85.7	242,241.61	5.6	184,971.06
Vermont	931,919	37,840.16	37,840.16		5.2	650,199.69	650,199.69		7.2	31.4	45,214.40	2.0	198,664.75
Virginia	3,708,379	34,776.95	24,007.24	10,769.71	2.4	1,047,556.49	1,041,143.33	6,413.16	22.8	42.4	919,044.76	32.4	1,724,183.67
Washington	3,057,934					1,624,910.95	1,621,554.13	3,356.82	26.9	65.7	377,324.24	13.8	1,059,055.63
West Virginia	2,013,405					1,623,778.26	1,623,778.26		32.0	69.1	155,296.17	3.5	234,330.57
Wisconsin	4,862,441	188,194.77	188,194.77		7.7	2,534,275.17	2,511,146.78	21,500.00	37.7	105.9	424,918.72	30.0	1,738,180.73
Wyoming	2,250,663	234,014.99	222,400.00		64.0	1,605,959.90	1,470,500.00	60,000.00	22.0	304.4	258,083.64	62.8	299,699.36
District of Columbia													
Hawaii	1,683,956					422,882.57	205,708.64	167,174.18	2.9	10.1	700,541.31	14.2	777,406.05
TOTALS	186,551,646	3,738,967.21	3,591,330.85	60,957.71	686.1	89,237,817.60	81,097,524.71	4,715,463.56	22.8	5,449.8	38,683,504.80	2,287.1	63,179,285.64

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS II—PROJECTS ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES

AS OF NOVEMBER 30, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	2,092,533					152,223.34	94,872.11	55,351.23	9.5	4.2	77,698.60	3.7	1,919,962.29
Arizona	781,794					11,448.25	11,448.25		43.7	1.0			770,345.75
Arkansas	1,687,084					175,032.27	142,569.74	32,462.53	4.2	4.4	115,784.17	3.9	1,428,730.09
California	3,901,839					434,891.45	352,400.79		11.6	4.2	560,761.59	13.0	2,988,676.62
Colorado	1,718,633	92,172.30	92,172.30		2.0	114,357.05	114,357.05		32.2	3.4	294,305.71	4.4	1,217,297.94
Connecticut	802,407					582,087.02	582,087.02		20.4	5.2	174,668.10	1.7	45,651.88
Delaware	454,772					199,363.90	199,363.90		27.6	2.0	27,621.00	.6	227,787.10
Florida	1,307,959					464,231.13	291,680.33	172,550.80	7.2	5.5	16,604.52	.4	999,674.15
Georgia	2,724,620					2,417.17	2,417.17		82.7	.1	274,334.09	5.5	2,447,868.74
Idaho	1,121,562	1,750.35	1,750.35		.6	202,508.04	201,178.14		11.4	5.1	122,231.05	1.7	796,402.46
Illinois	6,877,199					853,570.19	853,570.19		4.7	8.7	3,478,923.19	32.5	2,544,705.62
Indiana	4,818,165					24,444.50	24,444.50			.4			4,793,720.40
Iowa	2,815,585	113,047.37	105,250.00		4.1	732,270.99	688,200.00		31.5	16.2	396,200.00	19.8	1,625,935.00
Kansas	2,522,401					27,107.80	27,107.80		51.6	1.6	394,082.82	8.7	2,101,210.38
Kentucky	2,029,687					20,886.00	20,886.00			.6	129,191.65	2.7	1,979,609.35
Louisiana	1,457,148					306,912.55	306,912.55		29.0	5.8	115,739.54	5.5	1,034,495.91
Maine	842,479	39,866.89	39,866.89		.8	398,425.36	398,425.36		30.1	9.4	88,641.80	1.5	315,544.95
Maryland	891,132					16,788.76	12,638.76		39.6	.6			878,493.24
Massachusetts	4,136,382					1,552,083.41	1,505,836.55	40,000.00	8.6	8.6	39,786.17	.3	2,590,759.28
Michigan	4,457,679					446,070.00	443,920.00		46.2	8.6	1,273,651.00	14.4	2,740,108.00
Minnesota	3,419,102	311,734.16	311,734.16		19.6	765,318.40	765,318.40		43.4	35.3	315,375.68	10.6	2,017,673.76
Mississippi	1,744,669					101,857.51	56,296.81	45,560.70	16.0	2.4	64,673.51	4.0	1,623,698.68
Missouri	3,045,077					623,662.96	615,173.46		22.1	6.4	162,707.40	2.4	2,267,196.14
Montana	1,115,962					68,731.24	68,731.24		26.2	3.1	282,174.09	11.1	765,056.67
Nebraska	1,957,240	10,189.26	9,485.86		2.2	597,962.64	597,962.64		38.3	14.8	122,402.65	4.2	1,227,388.35
Nevada	500,051					50,304.27	50,304.27		89.5	1.2			449,746.73
New Hampshire	477,460					136,127.46	136,127.46		15.4	3.4	55,922.17	1.1	285,410.37
New Jersey	3,217,442					1,059,881.47	1,059,881.47		27.4	7.3	1,007,109.15	7.1	1,150,451.38
New Mexico	1,448,234					163,894.08	163,894.08		4.3	4.2	70,273.02	1.2	1,214,061.90
New York	7,837,865	20,000.00	20,000.00		.4	4,090,321.80	4,059,680.00		9.6	31.7	2,378,800.00	18.5	1,379,385.00
North Carolina	2,380,573	17,770.91	17,770.91		1.9	82,337.48	82,337.48		44.9	6.3	20,290.96	4.5	2,260,173.65
North Dakota	1,451,112	796.72	796.72		.6	48,305.94	48,305.94		26.9	6.5	147,780.50	6.4	1,254,228.94
Ohio	4,645,378	4,090.00	4,090.00		.1	1,009,350.00	919,375.00		21.0	12.6	636,785.00	9.4	3,085,128.00
Oklahoma	2,304,200					27,858.59	27,858.59		79.0	2.1	420,352.37	9.9	1,855,989.04
Oregon	1,526,724					449,951.08	449,951.08		8.9	6.8	98,615.55	4.4	973,157.37
Pennsylvania	5,416,051					492,077.73	408,985.64		14.7	10.7	1,617,453.24	29.4	3,389,612.12
Rhode Island	499,677										195,138.07	2.8	304,538.93
South Carolina	1,364,791					65,129.86	64,724.85	405.01	27.8	6.3	56,934.47	5.8	1,243,131.68
South Dakota	1,502,870	22,495.30	22,495.30		1.2	204,937.11	204,937.11		22.4	10.0	104,776.04	3.6	1,170,661.55
Tennessee	2,123,155					33,777.17	33,777.17		59.2	1.6	6,966.16	.5	2,082,411.67
Texas	6,061,006					401,389.14	375,004.21		3.7	23.9	1,074,339.12	52.4	4,611,162.67
Utah	1,048,677	27,156.85	27,156.85		.4	491,317.85	489,911.77		64.9	13.2	44,369.40	.8	487,238.98
Vermont	470,628	3,854.07	3,854.07		.6	248,762.39	247,743.78		8.9	7.3	120,719.59	3.2	98,310.56
Virginia	1,854,189	2,910.85	2,910.85		.5	174,693.30	165,932.81	8,760.49	24.7	4.3	609,618.21	4.9	1,075,727.13
Washington	1,977,571					237,676.38	237,676.38		18.9	6.3	257,515.07	4.8	1,382,379.55
West Virginia	1,342,270					66,508.64	66,508.64		22.6	.8	126,361.89	2.4	1,149,399.47
Wisconsin	2,431,220	131,355.96	131,355.96		4.4	618,952.59	618,952.59		26.3	16.5	191,766.11	3.6	1,489,145.34
Wyoming	1,125,332					144,250.17	143,750.17		20.9	2.7	181,572.92	6.8	800,008.91
District of Columbia	1,151,081	113,402.25	113,402.25		.8	672,952.26	672,952.26		17.2	2.5	202,689.34	1.3	162,037.15
Hawaii													
TOTALS	112,771,667	912,593.24	904,092.47		40.2	19,845,908.79	19,106,871.61	335,090.76	20.0	345.8	18,154,211.68	333.4	74,606,491.24

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BUREAU OF PUBLIC ROADS

VOL. 14, NO. 11 JANUARY 1934

CONSTRUCTING CIRCULAR TRACK FOR BITUMINOUS STABILITY TESTS

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

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BUREAU OF PUBLIC ROADS

G. P. St. CLAIR, *Editor*

Volume 14, No. 11 January 1934

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions

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STABILITY EXPERIMENTS ON ASPHALTIC PAVING MIXTURES

Reported by W. J. EMMONS, Associate Professor of Highway Engineering, University of Michigan[1]

IN ORDER to provide a background of service data for the development of laboratory strength tests for bituminous pavement surface mixtures, the Bureau of Public Roads built experimental sections of a wide range of compositions and observed their behavior under controlled traffic. These sections were constructed as a circular roadway 180 feet in mean diameter. The foundation was reinforced concrete. The test surfaces were 13 feet wide between an integral curb, 2 inches in height, at the inner edge and a concentric circle of experimental concrete sections which served as the outer curb. The roadway sloped uniformly from the outer to the inner curb with a drop of eight inches. The concrete foundation was finished to a high degree of smoothness, as it was desired to accelerate the tests, and foundation roughness is quite generally believed to assist in the resistance to displacement of the bituminous surface.

TWO SERIES OF TESTS CONDUCTED

Test sections were constructed and tested in two series. The first series was composed of 27 sections of asphaltic concrete and the second of 28 sections of sheet asphalt and 5 sections of asphaltic concrete. The surface course in all cases approximated 2 inches in depth. The mixtures were prepared at local paving plants and laid by contractors' forces. Surfaces were compressed with an 8-ton tandem roller. Due to the narrow width of the pavement, cross rolling was not possible. Delivery and placing of the mixtures was not always continuous, and for this reason the rolling was probably not uniform over the various sections. This may have resulted in lower densities in some instances, than would have been obtained in normal pavement construction. As far as possible, however, the procedure in construction was in accordance with current practice.

As the tests progressed, measurements were made on the surface mixtures as follows:

(1) The longitudinal displacement under traffic.
(2) The internal displacement at various depths.
(3) The temperature attained under prevailing climatic conditions.

ALL SECTIONS REMAINED STABLE UNDER WINTER TEMPERATURES

The sections were marked into thirds, with radial lines painted on the surface for use in measuring longitudinal displacement. These lines are shown in figure 2, and can be distinguished from the broader lines which bound the sections by the width of line and also by the two short lines normal to them near the inner curb. At each end of these radial lines, permanent metal reference plugs were set in the concrete. Wood screws, ⅝ inch in length were driven at 6-inch intervals along these lines. Before the tests were started, the position of each screw was recorded with reference to a wire stretched between the permanent plugs at the ends. Subsequently, as the tests progressed, the locations of the screws were similarly determined.

FIGURE 1.—LAYING SURFACE MIXTURE FOR STABILITY EXPERIMENTS.

The internal movement of the surface mixture was studied by observing the movement of short sections of ¼-inch brass rod set one above the other and with a short brass rivet on top. Four such installations were made on each section in the center of the strip over which traffic was to be concentrated. They were located 2 feet from the radial lines and were refereneed to the curb and to the wire which was stretched along the radial lines to measure longitudinal displacement.

Internal temperature of the pavement, was measured with thermo couples installed in several of the sections of the first series of tests. The scope of this phase of the investigation was considerably amplified in the second series. In general, temperatures were observed at 10 a.m. and 2 p.m., but some special work was done to investigate temperatures during a 24-hour period It was found that asphaltic mixtures are more liable to displacement under traffic during periods when high temperatures prevail. Many mixtures proved to be stable throughout the tests. Those which shoved badly under summer conditions were highly resistant to displacement during the cooler months. Under traffic of the type imposed, no mixture, however poorly constituted, was appreciably displaced during periods when the average air temperature was lower than 70° F. The results of the study of pavement temperatures have already been reported in detail.[2]

HEAVY TRAFFIC IMPOSED ON TEST SECTIONS

Loaded trucks equipped with solid rubber tires constituted the traffic. They traveled always in the same direction, and the inner wheels were kept in a path 2½ feet wide, as indicated by short longitudinal lines painted on the pavement. The speed was held as closely as possible to 12 miles per hour. In the first series of tests a 3-ton truck having a total weight of 15,600 pounds, and a rear wheel load of approximately

[1] Highway research specialist, Bureau of Public Roads.

[2] Temperature as a Factor in the Stability of Asphaltic Pavements, Public Roads, vol. 7, no. 2, April 1926

5,700 pounds was used at the beginning, but later another truck was added to accelerate the test. This second unit was a 5-ton truck, loaded to capacity, with a rear wheel load of about 7,900 pounds. Approximately 50,000 passages over the pavements were made between the start of the tests during the month of October and its discontinuance about the middle of the following August.

Only one truck was used in the second series of tests. The total weight was 17,800 pounds with a rear wheel load of 6,800 pounds. This truck made 64,000 trips over the pavements during two periods from August 28 to November 11, and from May 5 to October 15 of the following year. As the first series had shown that no displacement was to be expected during the cold months, traffic was suspended over the winter in this test. The west side of the roadway was located

FIRST SERIES OF STABILITY EXPERIMENTS DESCRIBED

In the first series of tests, the circular roadway was surfaced with 27 different compositions of asphaltic concrete. The intended variables were the relative percentages of fine and coarse aggregates, consistency of the asphalt cement, gradation of coarse aggregate and gradation of fine aggregate. Trap rock, graded from 1¼ inches to ¼ inch, Potomac River sand, and limestone dust were used respectively, as the coarse aggregate, the fine aggregate, and the filler in these mixtures.

At the time this series of sections was constructed, no paving plant equipped with weighing apparatus was available. All of the materials, even the asphalt cement, were proportioned by volume. This practice precluded strict adherence to the schedule of mixture compositions and probably also resulted in a lack of

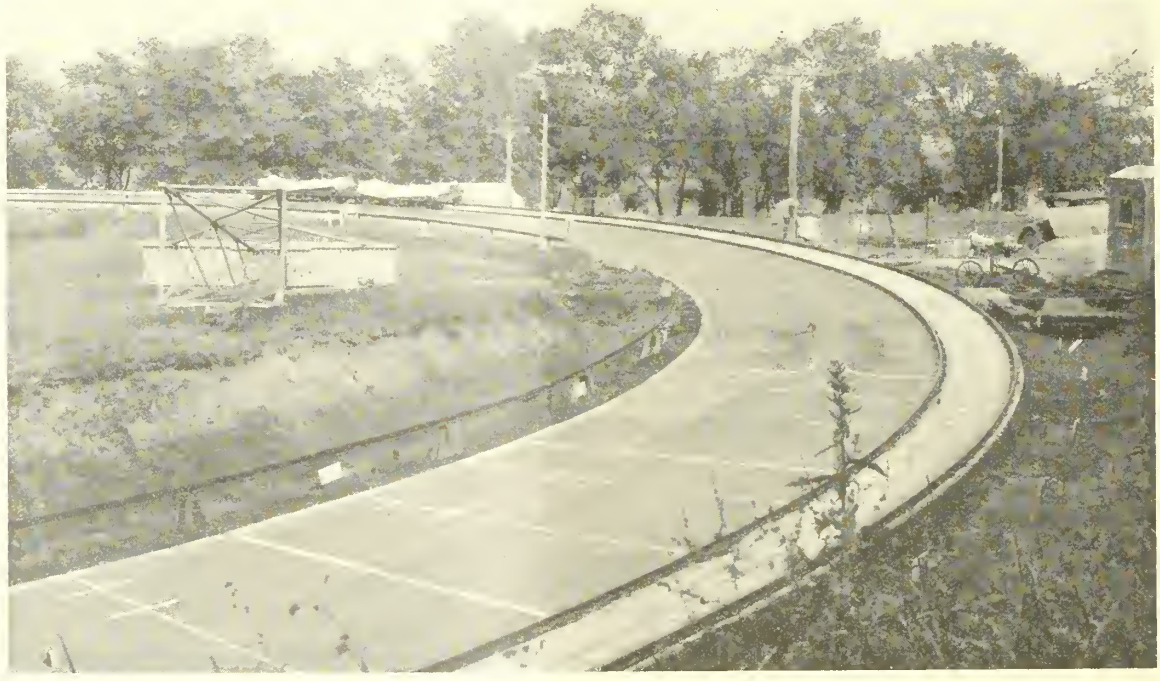


FIGURE 2.—SECOND SERIES OF STABILITY TESTS, SHOWING SECTION LINES AND EFFECT OF TRAFFIC ON TEST SECTIONS.

in a cut, the top of which, some 25 feet back, was fringed with trees and shrubs. These cast a shadow over some of the sections during the latter part of the day. In late September, this began about 1 p.m. on parts of sections 32 and 33. By 4:15, when traffic was discontinued for the day, sections 1, 2, 3, 4, 5, 7, 28, 29, 30, 31, 32, and 33 were either entirely shaded or merely spotted with sunlight which filtered through the foliage. The stability of asphalt mixtures is reduced as their temperature is increased, and it was determined in this investigation that at a given time the temperature of areas exposed to direct sunshine was higher than that of shaded areas. Therefore, it is true that certain of these mixtures were tested under slightly different conditions than the remainder for a part of each clear day. It is impossible to evaluate the effects of these temperature variations, but it is believed that they were not great enough to influence the behavior of the mixtures materially.

uniformity between the several batches composing each section. A number of mixtures, containing high percentages of coarse aggregate, gave compressed pavement surfaces of very open texture and a seal coat of asphalt cement and stone chips was applied to all sections. Table 1 shows the analyses of samples taken at the paving plant and the movement of the reference screw which displaced to the greatest extent under traffic. In table 2 the mixtures are grouped according to the consistency of the asphalt and the amount of coarse aggregate and the approximate gradation of the fine and coarse aggregates are shown.

RESULTS OF FIRST TEST NOT CONCLUSIVE

Most of the mixtures were satisfactory under traffic, the maximum displacement of any reference screw in one third of the sections being less than 1 inch. This is a negligible deformation which may merely reflect internal readjustment of the mixtures due to added

TABLE 1.—*Analyses of plant samples (percentages by weight) and maximum displacements of asphaltic concrete mixtures of first series of tests*

Mix no	Bitumen	Passing and retained on next smaller size: Retained on ½ inch	½ inch	1 inch	¾ inch	½ inch	¼ inch	No. 10	No. 40	No. 80	No. 200	Maximum size of displacement
	Per cent	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Inches*
1[1]	4.03	3.6	9.2	15.0	10.6	21.6	11.0	8.4	6.1	5.4	5.1	92.0
1A[1]	4.67	.....	3.4	8.5	19.5	29.5	9.7	10.2	6.3	3.9	4.3	
2	5.15	.....	1.6	4.8	30.9	24.0	8.5	8.5	6.0	5.6	5.0	62.5
3	4.83	.....	5.2	12.9	21.5	23.8	8.4	7.6	6.7	4.7	4.1	1.4
4	3.95	2.0	11.2	17.7	16.4	17.7	8.5	7.4	6.1	5.0	4.1	56.5
5	4.44	.....	11.1	15.8	11.1	18.9	8.1	12.1	8.7	6.0	3.7	24.2
6	4.59	2.1	10.5	12.1	14.6	17.7	8.2	10.4	9.3	6.6	3.8	42.5
7	5.06	4.0	7.0	12.6	10.9	19.1	10.5	12.5	9.3	5.7	3.3	23.8
8	5.41	.....	10.2	3.6	13.8	22.3	10.8	14.4	9.8	6.1	3.6	1.8
9	5.69	5.0	7.1	10.0	12.3	16.2	5.4	14.8	12.2	7.6	3.7	6.1
10	5.97	.....	4.1	13.5	11.5	19.5	6.8	14.7	12.2	7.7	4.0	4.6
11[1]	6.28	2.2	6.2	6.8	13.4	15.5	7.5	16.4	12.2	7.1	6.1	9.1
11A[1]	6.69	.....	6.4	6.9	12.0	20.0	7.5	15.2	12.7	7.8	4.8	
12	6.03	.....	6.1	9.2	12.7	15.3	6.4	15.6	14.7	9.6	1.4	.5
13	5.60	.....	7.2	7.2	11.0	8.6	8.8	24.3	14.4	7.6	5.3	1.1
14	6.02	.....	2.2	7.3	11.9	13.2	8.1	19.2	17.0	10.3	4.8	.4
15	7.43	.....	.....	1.8	9.2	14.1	8.4	20.6	21.8	11.7	5.0	2.4
16	6.32	.....	4.6	5.4	9.7	10.4	9.5	22.8	16.8	9.4	5.1	.3
17	5.04	7.0	5.7	12.4	9.3	13.6	11.0	15.1	9.6	6.6	4.7	.3
18	5.36	5.5	11.3	3.6	13.0	13.5	11.1	14.7	10.5	7.0	4.1	1.1
19	5.05	6.9	10.0	10.8	9.7	13.4	10.7	13.9	10.1	6.3	3.2	.7
20	6.31	.....	6.6	8.7	11.6	16.6	11.4	17.7	11.6	6.3	3.2	2.5
21	6.17	.....	6.2	12.2	9.4	15.1	6.8	15.6	14.6	9.5	4.4	1.1
22	6.20	.....	8.8	5.3	11.5	15.0	8.2	18.1	14.4	8.8	3.7	.9
23	5.78	.....	8.7	9.8	11.8	14.3	6.5	16.2	13.6	8.5	4.8	.5
24	4.92	.....	4.2	15.2	13.4	10.0	9.3	20.5	15.1	4.4	3.0	.4
25	5.03	4.4	7.4	11.4	9.7	12.5	11.7	18.1	12.8	4.1	2.9	.2
26	7.54	4.0	2.9	4.0	11.8	14.5	8.7	10.9	13.8	12.5	9.4	2.9
27	7.28	.....	1.6	7.2	17.1	11.8	4.9	6.9	17.2	17.0	9.0	7.9

[1] Analyses of batches mixed at different times.

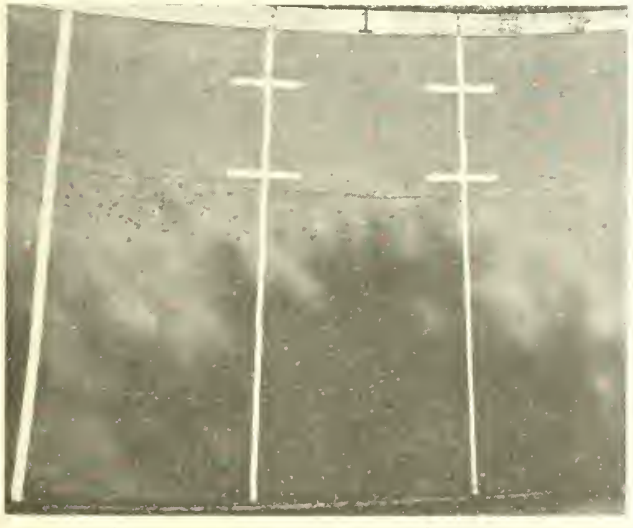


FIGURE 3.—CONDITION OF SECTION 3 ON COMPLETION OF TEST SHOWING FULL AREA OF SECTION.

compaction under traffic. A number of sections were displaced to extreme degrees, failure generally taking the form of rutting and progressive loosening of the bond between the coarse aggregate particles. The mixtures containing the higher percentages of coarse aggregate behaved in this manner. Subsequent examination of specimens taken from the less distorted areas of the sections which failed, revealed that the asphalt cement applied as the seal coat had penetrated to a considerable depth into these comparatively porous mixtures. It is probable that this excess of bitumen contributed largely to their instability. It was further noted that a large part of the coarse aggregate particles of these disintegrated areas were almost or entirely free from a coating of bitumen. This may have resulted

TABLE 2.—*Composition of plant samples (percentages by weight unless otherwise shown) and displacement of asphaltic concrete mixtures of first series of stability tests*

Mix no	Maximum screw movement	Bitumen	Coarse aggregate retained on no. 10 sieve: By weight	By volume	Passing and retained on next smaller size: ½ inches	¼ inches	1 inch	¾ inch	½ inch	¼ inch	No. 10	No. 40	No. 80	No. 200
	In.	*Pct.*	*Pct.*	*Pct.*	*Pct*	*Pct*	*Pct.*	*Pct*	*Pct*	*Pct*	*Pct*	*Pct*	*Pct.*	*Pct.*
MIXTURES CONTAINING 15 PENETRATION ASPHALT														
16	0.3	6.32	40	33		12	14	21	26	21	12	31	17	10
23	0.5	5.78	51	44		17	19	23	28	13	38	31	20	11
12	0.5	6.03	50	42		12	19	25	31	13	35	33	22	10
19	0.7	5.05	61	54	11	16	18	16	22	17	11	30	19	10
8	1.8	5.41	60	53	.....	17	6	22	37	18	12	29	18	11
MIXTURES CONTAINING 55 PENETRATION ASPHALT														
25	0.2	5.03	57	49	8	13	20	17	22	20	17	31	11	8
24	0.1	4.42	52	43	.....	8	29	26	19	18	18	35	10	7
22	0.9	6.20	49	41	.....	18	11	24	31	17	10	32	20	8
18	1.1	5.36	58	51	9	19	6	22	24	20	41	29	19	11
15	2.4	7.43	34	28	.....	.....	5	28	42	25	31	37	20	9
20	2.5	6.31	55	47	.....	12	16	21	30	21	16	30	16	8
26	2.9	7.51	46	38	9	6	9	26	31	19	23	30	27	20
27	7.9	7.28	43	35	.....	4	17	40	27	12	11	34	31	18
11[1]	9.4	6.18	52	44	2	12	13	21	35	11	38	30	19	13
7	23.8	5.26	64	57	6	11	20	17	30	16	10	30	19	11
1	56.5	3.95	73	66	3	15	24	22	21	12	33	27	22	18
MIXTURES CONTAINING 65 PENETRATION ASPHALT														
14	0.4	6.02	43	36	.....	6	17	28	31	19	37	33	21	9
3	1.4	4.83	71	64	.....	7	18	30	33	12	33	29	20	19
10	4.6	5.47	55	48	.....	7	21	21	36	12	38	32	20	10
6	42.5	4.59	65	58	3	16	19	22	27	13	31	31	22	13
MIXTURES CONTAINING 75 PENETRATION ASPHALT														
17	0.3	5.24	60	52	12	10	21	15	23	19	42	27	18	13
21	1.1	6.17	50	42	.....	13	21	19	30	14	35	33	22	10
13	1.1	5.60	43	36	.....	17	17	25	21	20	47	28	15	10
9	6.4	5.69	56	50	9	13	18	22	28	10	38	32	20	15
5	24.2	4.44	65	58	.....	17	24	17	29	13	10	28	20	12
2	62.5	5.15	70	62	.....	2	7	44	35	12	34	24	22	20
MIXTURE CONTAINING 86 PENETRATION ASPHALT														
1[1]	92.9	4.35	71	64	3	9	16	21	36	15	37	25	19	19

[1] Average of analyses of batches mixed at different times.

from the abrasive action during the progressive loosening of the bond between the stone particles, but there is also the possibility of poor adhesion of the asphalt to the coarse aggregate.

The number of factors which affected, in varying degrees, the behavior of these 27 mixtures under traffic, makes it impossible to build up a general theory which will satisfactorily explain the observed results. There are, however, a few interesting observations which can be made:

(1) Mixtures which contained as much as 64 percent by weight (approximately 57 percent by volume) of coarse aggregate were, with one exception, badly displaced by the traffic. This behavior probably was due, as previously suggested, to the increased bitumen introduced by application of the seal coat, or to insufficient mortar to properly reinforce the interlocking of the coarse aggregate. This latter possible cause may have been supplemented by a rather poor degree of adhesion or to a loss of adhesion of the asphalt to the coarse aggregate particles.

(2) Mixtures which were highly and equally stable were prepared with asphalts of 45, 55, 65 and 75 pene-

tration. The amount of bitumen in the mixture and other details of proportioning are undoubtedly far more important factors in their effect upon stability than is the consistency of the asphalt cement.

(3) Most of the mixtures contained fine aggregates which were considerably coarser than usual aggregate specified for sheet asphalt construction. The fine aggregates of those mixtures which were most resistant to traffic displacement did not carry high percentages of filler. The average fine aggregate gradation of the five mixtures prepared with 45 penetration asphalt, all of which were highly stable, was substantially as follows:

Passing no. 10, retained no. 40 sieve	40
Passing no. 40, retained no. 80 sieve	31
Passing no. 80, retained no. 200 sieve	19
Passing no. 200, retained	10
Total	100

(4) There is no indication that the gradation of the coarse aggregate affected the behavior of the several mixtures. If gradation is of significance within the range of sizes used, its effect is obscured by the greater importance of other factors.

SECOND SERIES OF STABILITY EXPERIMENTS INITIATED

The difficulties encountered in correlating the results of the first series indicated the necessity of tests with a smaller number of variables involved. A second group of sections was planned, the major portion of which were sheet asphalt mixtures. Each was to be of different composition and to be laid 2 inches thick directly on the smooth concrete foundation without the customary intermediate or binder course. A few sections of asphaltic concrete were included to obtain direct comparisons with sheet asphalt. Dense asphaltic concrete mixtures were designed to avoid the necessity for a seal coat. Table 3 shows the schedule of mixtures. Potomac River sands typical of those used locally were used with the exception of a very fine sand imported for mixtures 1 to 5. This fine sand was blended with the medium sand used in mixtures 11 to 28. Trap rock was not available for the asphaltic concrete mixtures and a hard limestone was used. The asphalt cements were refined from Mexican crude oil and, with the exception of those which conformed to standard grades, were specially prepared, presumably from the same stock. Their test characteristics are given in table 4. The 42-penetration asphalt was drawn from the storage tanks at the paving plant. The other asphalts, of which only small amounts were needed, were heated in small auxiliary kettles.

FAILURES EXPLAINED BY FAULTS IN PROPORTIONING OR IN MATERIALS

One truck load was sufficient to surface each section of approximately 25 square yards. The frequent changes in formula, with the necessary substitution of aggregates and asphalts, resulted in considerable delay and difficulty in proportioning and in the control of temperatures. The 33 mixtures were produced in two working days. Large samples of each mixture were taken and their analyses are shown in table 5. Some discrepancies exist between the mixtures as planned and the analyses. Those which involve the percentage of filler are doubtless due to variations in the content of the Potomac River asphalt sand passing the no. 200 sieve. This sand was used from the contractor's stock. Sheet asphalt section no. 16 is an outstanding example of such discrepancy. No filler was added to this mixture, but repeated tests on samples yielded between 9.0 percent and 10.0 percent passing the no. 200 sieve.

Although the mixtures do not conform in every detail to those originally outlined, they do cover a wide range of composition and it is quite certain that each section is uniform. It is regretted that mixtures typical of the high filler combinations which have been popular in some quarters were not included in this series. Mixture 19 carried a large quantity of dust, the effect of which, however, was obscured by other factors which are discussed later.

Traffic was started on the pavements in late August as soon as measuring devices had been installed. Dis-

TABLE 3.—*Schedule of mixtures for second series of stability experiments*

Group	Mixtures or section numbers	Percentage of bitumen	Percentage of filler	Type of sand	Coarse aggregate	Type of asphalt	Penetration of asphalt
A	1 to 5	Variable	Constant	Fine	None	Steam-refined	Constant.
B	6 to 10	do	do	Coarse	do	do	Do.
C	11 to 15	do	do	Medium	do	do	Do.
D	16 to 19 and 13	Constant	Variable	do	do	do	Do.
E	20 to 23 and 13	do	Constant	do	do	do	Variable.
F	24 to 28	do	do	do	do	Blown	Do.
G	29 to 33	Variable	do	do	Limestone	Steam refined	Constant.

TABLE 4.—*Analyses of asphalt cements used in second series of tests*

Type	Mexican, steam refined					Mexican, blown				
Sections in which used	20	(1)	21	22	23	24	25	26	27	28
Specific gravity, 25/25° C.	1.052	1.052	1.043	1.035	1.034	1.037	1.030	1.024	1.020	1.022
Flash point, ° C.	324	299	296	299	296	258	227	235	245	243
Penetration, 0° C., 200 g, 1 min.	11	15	16	19	21	17	22	27	28	34
Penetration, 25° C., 100 g, 5 sec.	31	42	55	63	72	44	50	59	67	81
Penetration, 46.1° C., 50 g, 5 sec.	119	175	234	(2)	(2)	137	126	195	250	(2)
Softening point, ° C.	61.9	55.9	53.4	51.4	49.0	60.6	64.4	55.3	53.2	50.3
Ductility, 25° C., cm.	77	95	87	100+	100+	27	13.5	55	65	53
Volatilization, percentage change in weight, 50 g, 5 hr., 163° C.	+0.002	−0.004	−0.021	+0.004	−0.016	−0.062	−0.052	−0.110	−0.109	−0.146
Penetration of residue from volatilization, 25° C., 100 g, 5 sec.	27	39	50	56	62	41	46	55	61	73
Total bitumen, soluble in CS_2 percent	99.77	99.65	99.62	99.56	99.59	99.72	99.63	99.31	99.53	99.50
Organic material, insoluble, percent	0.15	0.20	0.24	0.26	0.20	0.20	0.25	0.31	0.31	0.28
Inorganic material, insoluble, percent	0.08	0.15	0.14	0.18	0.21	0.08	0.12	0.38	0.16	0.22
Bitumen insoluble in 86° B. naphtha, percent	33.07	32.21	30.03	25.10	24.69	35.09	36.24	28.13	30.86	29.35
Fixed carbon, percent	17.18	17.75	15.40	13.65	14.09	15.35	15.00	13.30	13.42	12.60

[1] 1 to 19, 29 to 33, inclusive.
[2] Too soft for test.

placement began immediately in a number of the sections, but as the temperature declined during the autumn months the effect of traffic became increasingly less. The test was discontinued during the winter and recommenced in May and completed in October. Before traffic was resumed in May, a core 4 inches in diameter was punched from a location in each section which had received full compression in construction but which had not been traveled over by the truck. The densities of these plugs were regarded as representative of the degree of compression of the respective sections and are referred to later in this report.

After completion of the traffic tests, 10 slabs or more approximately 24 inches square were cut from each section for use in laboratory studies. These specimens were taken within the area of traffic as well as near the curbs and between the wheel tracks. At the same time the internal displacement rods were excavated, their positions recorded, and movements computed.

The final longitudinal displacements of the several sections are shown in the last column of table 5. The displacement of each section was taken to be the total movement of 25 screws in a radial line derived by taking half of the total movement of 50 screws in two lines. Figures 3 to 6 show the condition of the several sections at the end of the test. These pictures were taken from an elevated platform after the section and guide lines had been repainted. Displacements occurred in the direction of traffic, and they developed to a greater degree under the outer truck wheels. Instability was evidenced more by rutting with lateral displacement than by the formation of transverse waves. Some movement occurred in every section, but in many it was very slight. All failures can be readily explained by faults in the proportioning or quality of asphalt.

The degree of displacement appeared to be fairly uniform through the depth of the surface course. The sections of brass rod closely maintained their original relative positions with the exception of those in the highly plastic mixtures. In a few instances there were somewhat greater movements in the upper inch, but the actual distance moved was but slightly greater than that immediately at the foundation. Sometimes the movement at lower levels appeared a little greater than at the top, but this difference also was negligible. Typical diagrams of the migration of the reference rods are shown in figure 7. There was no bond between the surface course and the smooth foundation. It appears that the use of the customary stable binder course, which provides a positive bond with the surface course, might have improved the behavior of some of these mixtures. It is doubtful if the highly plastic mixtures would have been benefited.

The primary object of the experiment was to supply specimens of known relative degrees of stability for the subsequent development and correlation of laboratory stability tests, but the results provide an opportunity to study the practical effects of some variations in materials and compositions.

SHEET ASPHALT SECTIONS DISCUSSED

Three sand combinations of widely different gradations were used as shown in table 3. Five mixtures were laid with each sand combination, varying only the bitumen content. It was the intention to commence with a very lean mixture, and gradually increase the bitumen content until a condition of excessive richness was reached. Mixtures 1 to 5 with fine sand, 6 to 10 with coarse sand, and 11 to 15 with medium sand constituted these groups. The fine-sand and coarse-sand groups are especially uniform in aggregate composition and illustrate well the influence of changes in bitumen content. In each group resistance to displacement increased with decrease in bitumen content.

No material decrease in the stability of mixtures 1 to 10 was evident until the bitumen was increased to the

TABLE 5.—*Composition, density, voids, laying temperature and displacement under traffic of sheet asphalt sections*

Section or mixture	Bitumen	Passing and retained on next smaller size								Specific gravity of aggregates	Specific gravity of core from pavement	Theoretical maximum specific gravity	Pavement voids	Laying temperature (°F.)	Displacement	
		Retained on no. 10	No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200						Greatest screw movement	Total movement of 25 screws[1]
	Percent by weight	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*	*Percent by weight*				*Percent*		*Inches*	*Inches*
1	10.2	0.2	1.8	4.3	3.8	9.3	16.3	16.0	25.6	12.5	2.675	2.055	2.312	11.0	400	0.1	1.4
2	10.9	.2	2.1	4.3	4.2	9.2	15.7	18.9	22.0	12.5	2.674	2.048	2.290	10.7	275	.6	4.0
3	12.0	.3	2.0	3.8	3.5	8.3	14.9	19.8	23.3	12.1	2.660	2.086	2.249	7.2	450	.5	2.7
4	12.6	.7	3.7	5.6	4.9	9.9	14.8	16.3	19.1	12.4	2.681	2.104	2.243	6.2	325	32.1	199.8
5	13.9	.3	2.9	5.0	3.8	8.4	14.4	14.2	24.2	12.9	2.666	2.107	2.199	4.0	335	86.0	311.2
6	7.9	6.3	16.7	12.5	7.6	10.8	10.9	7.3	7.5	12.5	2.688	2.150	2.395	10.2	350	.3	1.6
7	8.7	5.5	13.4	12.9	7.8	11.8	12.7	7.3	8.6	11.3	2.674	2.100	2.359	11.0	375	1.7	8.3
8	9.0	6.5	16.6	11.6	7.0	10.4	11.2	8.0	7.9	11.8	2.674	2.175	2.349	7.4	325	5.3	23.4
9	11.1	4.5	14.7	12.7	8.2	11.3	10.8	7.3	7.2	12.2	2.671	2.202	2.283	3.5	300	70.0	379.0
10	12.2	3.7	12.8	12.9	8.0	11.4	12.3	6.4	8.0	12.3	2.666	2.198	2.246	2.1	300	115.9	448.9
11	8.2	.2	3.3	6.0	5.9	13.1	19.5	12.8	18.3	12.7	2.674	2.018	2.374	15.1	280	.7	6.7
12	10.0	.2	4.1	7.4	6.2	13.0	18.7	10.4	16.4	13.6	2.674	2.102	2.317	9.3	325	5.7	56.1
13	10.4	.3	2.6	5.8	5.4	12.2	19.0	11.0	16.8	16.5	2.666	2.150	2.299	6.5	375	4.3	32.8
14	11.5	.2	2.6	5.7	5.5	13.2	17.9	13.7	18.2	11.5	2.676	2.075	2.273	8.7	325	13.2	37.6
15	12.3	.3	3.0	6.4	5.8	13.4	19.6	10.6	16.3	12.3	2.666	2.187	2.243	2.4	340	37.7	267.4
16	9.7	.4	4.3	8.7	7.0	14.8	19.8	10.7	14.7	9.9	2.656	1.999	2.314	13.6	375	.6	7.1
17	9.8	.2	3.8	7.9	7.0	14.5	19.2	11.3	16.1	10.2	2.674	1.990	2.323	14.4	325	1.5	15.6
18	10.1	.2	2.2	5.2	5.1	12.8	18.4	12.9	18.1	15.0	2.688	2.170	2.323	6.5	350	4.5	46.9
19	10.9	.1	2.3	4.5	4.4	10.9	15.2	11.3	16.7	23.7	2.696	2.230	2.304	3.1	280	17.0	140.8
20	9.9	.3	2.3	4.6	4.8	12.7	18.4	11.0	17.0	16.0	2.666	2.010	2.315	13.2	325	.6	2.8
21	10.0	.1	2.2	5.2	5.1	12.3	19.3	11.8	18.2	15.8	2.666	2.091	2.307	9.4	340	.5	5.1
22	9.7	.2	4.1	8.1	6.8	13.7	18.8	10.8	15.0	12.8	2.666	2.091	2.313	8.6	320	18.7	109.0
23	10.4	.2	3.6	6.7	6.3	13.7	18.1	13.0	15.9	12.1	2.668	2.073	2.291	9.5	355	9.1	71.1
24	10.0	.1	3.1	5.6	5.5	12.9	18.3	13.3	15.4	15.8	2.682	2.070	2.315	10.6	340	2.7	28.1
25	9.9	.1	3.6	6.7	6.0	13.9	18.0	13.7	16.3	11.8	2.669	2.009	2.306	12.8	310	4.9	[2] 48.4
26	10.0	.2	2.8	6.3	6.1	13.9	18.3	12.8	17.3	12.9	2.664	1.938	2.296	15.6	350	7.7	82.9
27	10.5	.2	2.8	6.7	6.4	13.6	19.1	10.6	16.5	13.6	2.666	1.940	2.280	14.9	290	36.3	244.2
28	9.7	.2	3.1	7.1	6.2	13.8	18.8	10.7	15.2	15.2	2.681	1.950	2.316	15.9	340	10.5	87.0

[1] Averaged from movement of 50 screws in two lines.

[2] Approximate as section partly disintegrated under traffic.

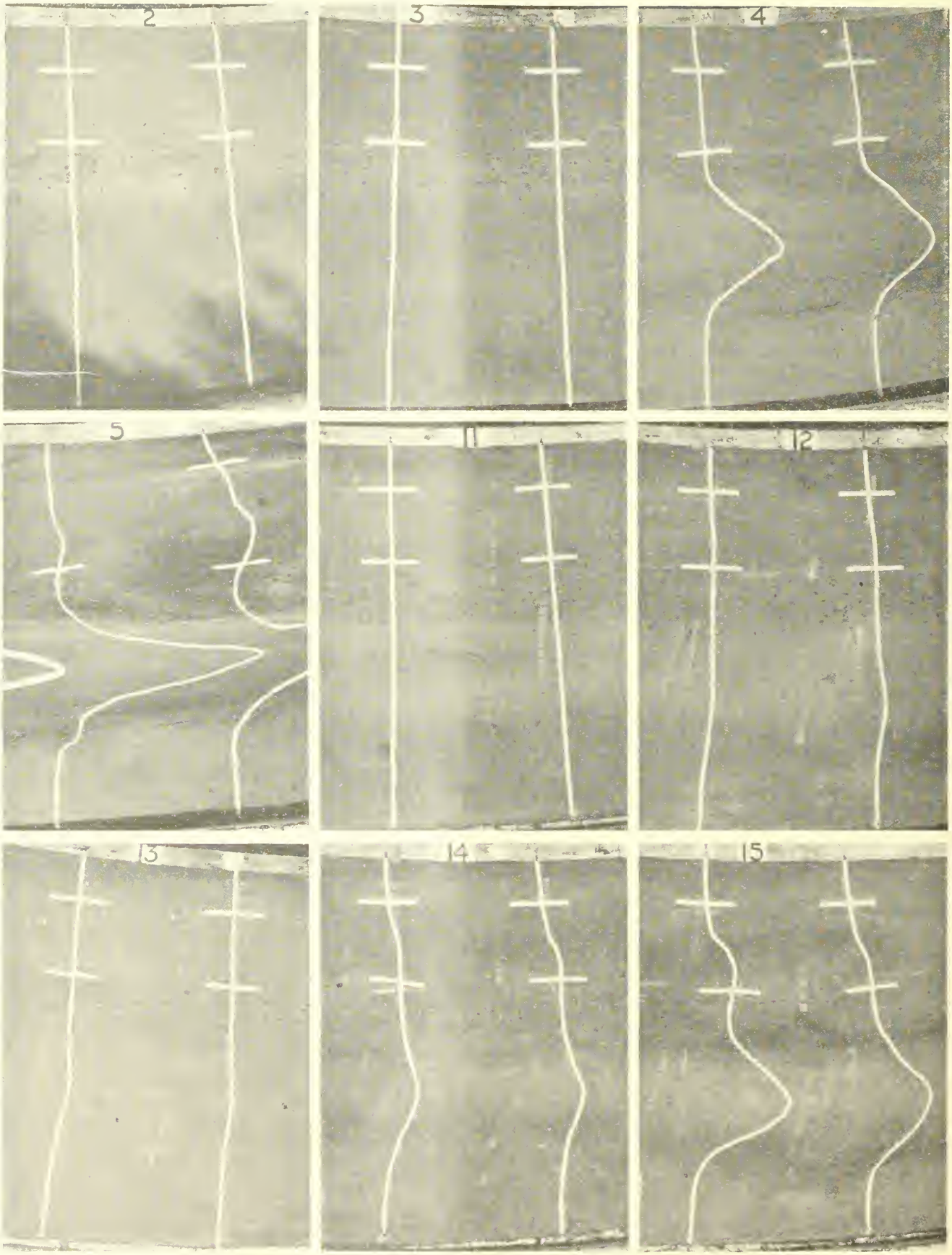


FIGURE 4.—CONDITION OF TEST SECTIONS OF SECOND SERIES ON COMPLETION OF TESTS.

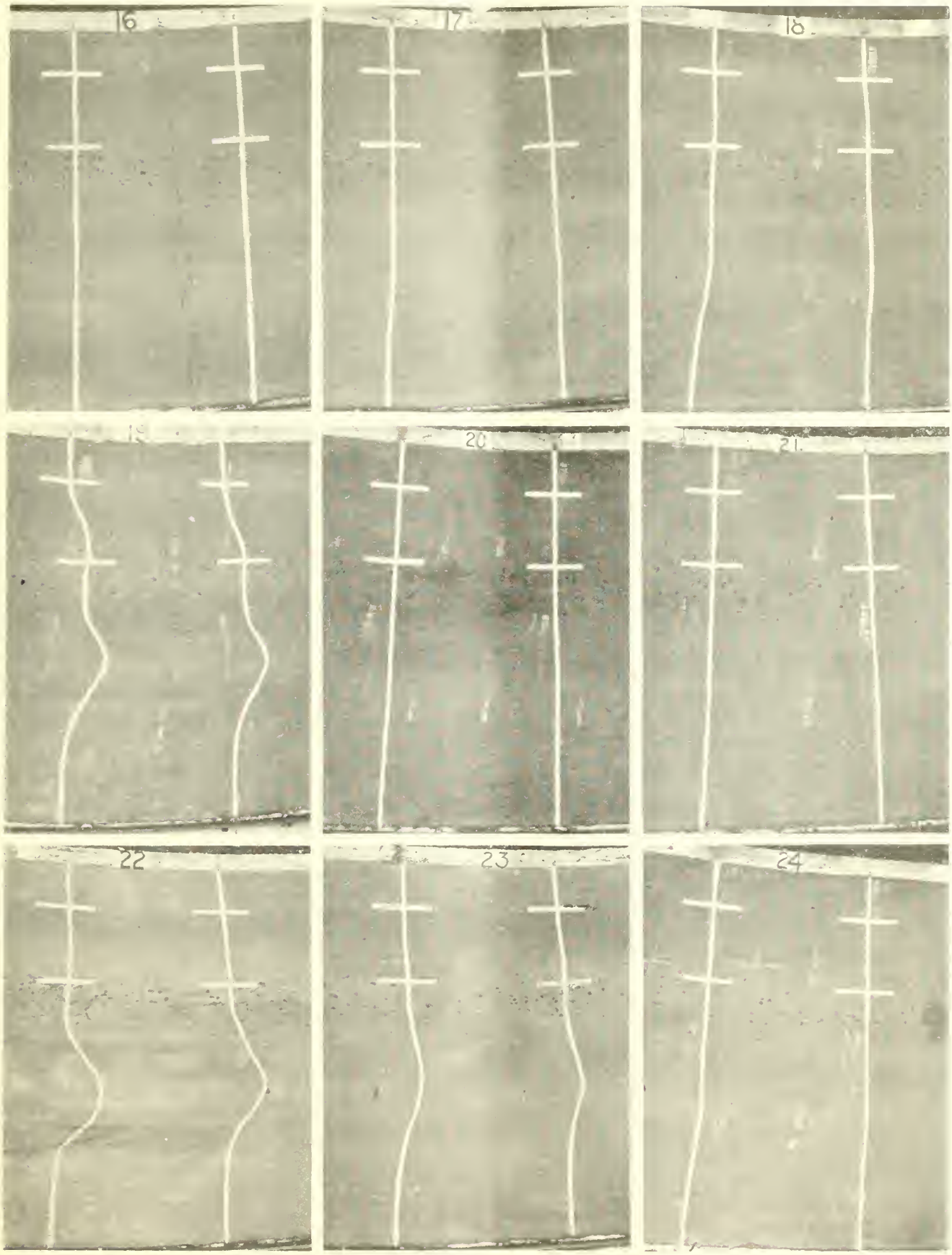


FIGURE 5.—CONDITION OF TEST SECTIONS OF SECOND SERIES ON COMPLETION OF TESTS.

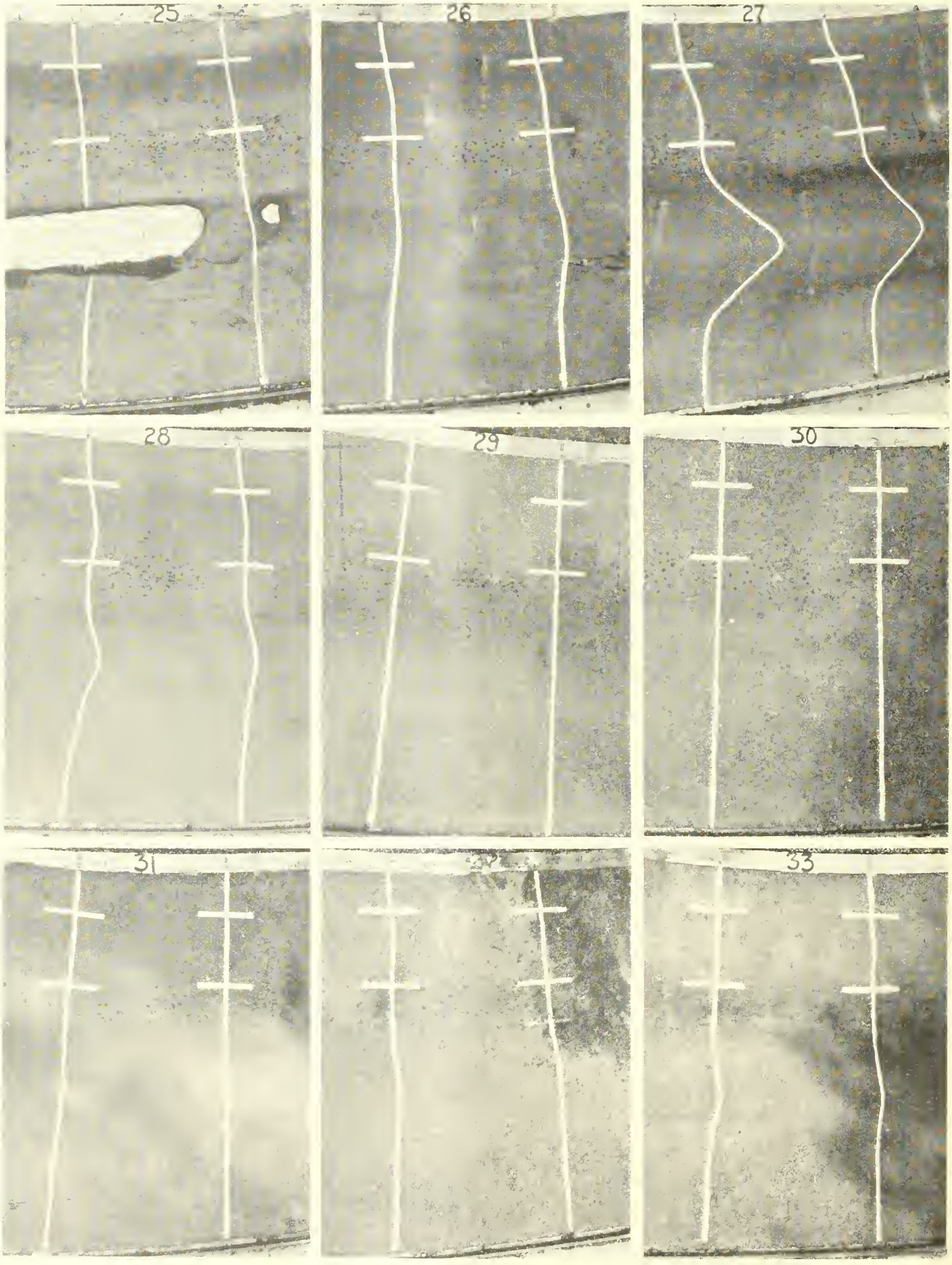


FIGURE 6. CONDITION OF TEST SECTIONS OF SECOND SERIES ON COMPLETION OF TESTS.

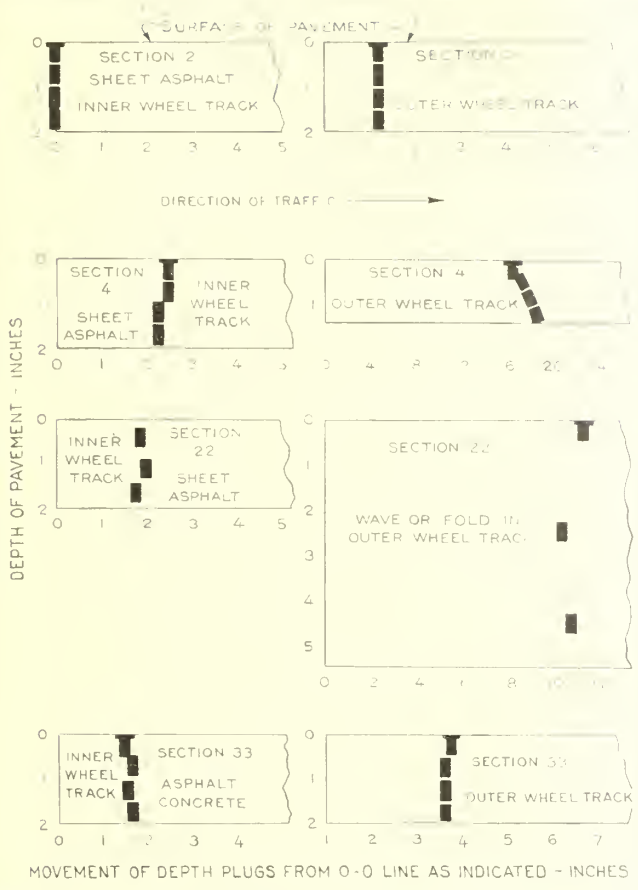


FIGURE 7.—DISPLACEMENT OF MIXTURES AT VARIOUS DEPTHS BELOW SURFACE OF PAVEMENT.

percentages used in mixtures 4 and 9. These two mixtures were so unstable as to indicate a critical percentage somewhere between 12.0 percent and 12.6 percent with the fine sand, and between 9.0 percent and 11.1 percent with the coarse sand. The medium-sand mixtures, numbers 11 to 15, exhibit the same tendencies, but for some reason the aggregate of mixture 13 proved to contain more than the intended percentage of fine material passing the no. 200 sieve and its behavior is somewhat out of line. It is more than probable that the high percentage of material in this mixture passing the no. 200 sieve results from a relatively high silt content of the sand stock, as those mixtures prepared immediately preceding it (numbers 21 and 24) exhibit similar compositions. The slightly greater stability of mixture no. 13 as compared with no. 12 must therefore be attributed to more favorable characteristics of its aggregate. Results obtained in these three groups indicate that, if properly proportioned, satisfactory sheet asphalt mixtures can be prepared with either fine, medium or coarse graded sand.

Mixtures 16 to 19 and including mixture 13 were intended to develop the effects of variations in the amount of filler, the percentage of bitumen being held constant. Analyses of the mixtures do not check with the plant formulas for the respective mixtures, not only in bitumen content but in the percentage of aggregate passing the no. 200 sieve. This is particularly noticeable in the case of mixture 16 to which no dust was added. The material passing the no. 200 sieve in this mixture represents silt in the sand supply. Mixture 13 also contained 3 percent to 4 percent more material passing the no. 200 sieve than should have been derived from the amount of limestone dust which was added and from the normal sand supply. Although these mixtures proved progressively less stable as the filler was increased, the variations in bitumen content and in the nature of the filler material make it impossible to attribute the difference in service behavior to increased filler alone.

Illustrating this point, comparison of mixtures numbers 18 and 21 show them to be almost identical in bitumen content and in aggregate gradation, although it is known that considerably more of the filler material of mixture 21 is silt than is the case in mixture 18. Although containing bitumen of somewhat softer consistency, mixture 21 seems to have been the less compressible and was certainly the more stable of the two mixtures. In this series it is true that increased percentages of the limestone dust filler accompanied by increased bitumen resulted in decreased resistance to traffic displacement. However, it is apparent that the nature and characteristics of the filler material have an effect on the behavior of the mixture. It will be shown later that voids in the mixture and in the aggregates are significant functions which are dependent upon interrelationships between the sand, filler, and bitumen. Therefore a general conclusion to the effect that increased filler is likely to cause instability is not justified, for the most desirable percentage can be determined only by studies of the several constituent materials in various combinations.

Asphalts similar in all characteristics except degree of hardness were used in mixtures 13 and 20 to 23. Here again the effects of the controlled variable are obscured by lack of uniformity in other characteristics. The aggregate composition of mixtures 20, 13, and 21 are closely similar. These mixtures contain asphalts of 31, 42, and 55 penetration respectively and the mixtures may be properly compared with regard to this variable. Between sections 20 and 21, containing asphalt varying in penetration by 24 points, no marked difference in performance was evident. Both sections behaved excellently. Mixture 13 displaced slightly although its bitumen was harder than that in section 21. Its slightly greater richness as well as the presence of an extremely plastic mixture adjacent to it probably explains the small difference in behavior. Mixtures 22 and 23 displaced somewhat more than those just discussed. Comparing mixture 22 with mixture 20 it is found that they differ only in percentages of particles passing the no. 200 sieve and in penetration of asphalt. The same is true of mixtures 23 and 13.

In spite of the slight variations in the bitumen and filler contents of the mixtures in this series and the lack of a consistent relationship between consistency of asphalt and displacement of the mixture, the much greater displacements in sections 22 and 23 are undoubtedly due to the higher penetrations of the asphalts used in these sections. As compared with the material used in section 21, the asphalts in sections 22 and 23 had penetrations at 25° C. (77° F.) higher by only 8 and 17 points, respectively. However, at 46.1° C. (115° F.) the asphalt in section 21 had a penetration of 234 while the asphalts in sections 22 and 23 were too soft to test under standard conditions at this temperature. None of these sections displaced under traffic until the air temperatures became quite high and the pavement temperatures much higher. The greater movements in sections 22 and 23, therefore, may be attributed directly to the softening of the

asphalt to a greater extent under the summer heat, resulting in mixtures less resistant to the action of traffic, than was the case with asphalts which were but slightly harder as measured by the penetration at 25° C. (77° F.).

DISINTEGRATION OBSERVED ON BLOWN ASPHALT SECTIONS

The sections containing the blown asphalts (mixtures 24 to 28) behaved, as far as displacement is concerned, very much as did those sections containing steam refined asphalts. The influence of the lower percentage of bitumen and possibly that of the higher content of material passing the no. 200 sieve in promoting stability are apparent. The softer asphalts may be a little less effective in resisting displacement, although the virtually identical behavior of sections no. 26 and no. 28 with asphalts of 59 and 81 penetration respectively, indicate that the hardness of the binder is certainly not the most potent factor affecting stability.

The mixtures containing blown asphalt showed an unexpected tendency to disintegrate under traffic and this tendency was confined solely to these mixtures. Disintegration was most marked in section 25, where the binder became brown and lifeless; the mixture crumbled, and wore down to the foundation as the test progressed. It is perhaps significant that the asphalt of this mixture had the lowest ductility, 13.5 cm, of any of the group. Section 24 showed slight indication of deterioration in a single small crack, but the other three of the group were beginning to show distress when the test was terminated.

The mixtures of this group were apparently not compacted to the degree to which they were susceptible. Although this may have rendered them liable to some displacement, the fact remains that perfectly stable sections in other groups were constructed with equally high voids. The outstanding indication is that only the blown asphalt mixtures showed deterioration through loss of binding power of the asphalt.

It is obvious that each of these groups of sheet asphalt mixtures had a certain resistance to displacement, which apparently depended to a great extent upon the amount of bitumen present. The limit of resistance was reached sooner on some sections than on others. The same medium sand combined with similar percentages of filler has long been used in the construction of Washington streets, and has successfully carried from 10.5 percent to 11.5 percent by weight of bitumen, which indicates the unusually severe conditions of these tests.

AGGREGATE VOIDS DETERMINED BY VARIOUS METHODS

The voids in the compacted mineral aggregate are considered by many to exert a controlling effect on mixture design. A considerable investigation was conducted, therefore, on the aggregate voids of these mixtures and the degree to which they were filled with bitumen. Determinations of aggregate voids were made in four different ways, as follows:

(1) By computation based on test results from cores taken from the various sections and representing the compression received by each mixture in construction.

(2) By direct determination using the cone method [3] on samples of aggregates extracted from the mixtures.

(3) By direct determination, using small cylinders in the vibrator developed by the Bureau of Public Roads [4] for this work, on samples of aggregates extracted from the mixtures.

(4) By computations using laboratory determinations on compressed specimens prepared by the method of tamping and compression prescribed for the Hubbard-Field stability test.[5]

Figure 8 and table 6 show the percentages of voids in the aggregates of the various mixtures as determined by these four methods. It will be noted that compaction by the Hubbard-Field method nearly always results in the lowest voids in the aggregate. There are a few exceptions to this where sufficient bitumen was present to prevent ultimate compaction of the aggregate particles. The use of the vibrator resulted in percentages of aggregate voids which were generally higher than those obtained by the Hubbard-Field method, and, in general, lower than those found by the cone method or in the pavement as actually compressed. The compaction obtained by the vibrator method was sufficient to wedge the aggregate particles tightly in the container and they had to be loosened by a pointed metal rod in removing.

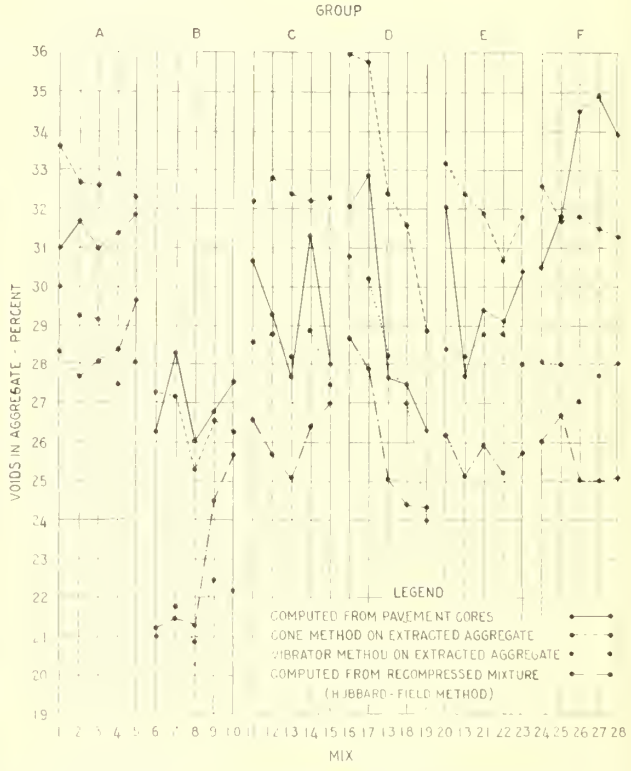


FIGURE 8.—VOIDS IN AGGREGATES OF SHEET-ASPHALT MIXTURES AS DETERMINED BY FOUR METHODS.

[3] A small metal cone of approximately 180 cc capacity was filled with the aggregate in four equal increments. After each addition of aggregate, the cone was beaten for 5 minutes on a brass plate mounted on a rigid base. After the last increment was added, beating was continued until no decrease in volume could be obtained. Percentage of voids is computed from weight of the known volume of aggregate contained in the cone and the specific gravity of the aggregate.

[4] Researches on Bituminous Paving Mixtures, Public Roads, vol. 7, no. 10, December 1926, p. 205.

[5] The material for molding the Hubbard-Filed specimens is first heated indirectly by means of an oil bath in order to prevent overheating, to the molding temperature of 250° F. The amount necessary to form a compacted specimen one inch high is then placed in the warmed cylinder and given 60 blows with the blade tamper and 15 blows with the blunt tamper. The cylinder containing the tamped material is then placed in a suitable coutainer, the plunger inserted and placed under the head of the Hubbard-Field machine and pressure of 3,000 pounds per square inch is applied. Water is then poured in the container and around the cylinder in order to cool the specimen, maintaining the pressure while cooling. After remaining under pressure in water for 5 minutes the sample is removed from the mold, specific gravity determined by displacement method and percentage of aggregate voids computed.

The following formula is used:

$$\text{Percentage of voids in aggregate of compacted mixture} = 100 - \frac{MS}{A}$$

where M = deusity of compacted specimen.
S = percentage by weight of aggregate in mixture,
A = specific gravity of aggregate.

TABLE 6.—*Voids in aggregates of sheet-asphalt mixtures and degree to which voids were filled with bitumen*

Mixture or section number	Percentage of voids in aggregates				Percentage of aggregate voids filled with bitumen, assuming mixtures compacted to densities corresponding to voids of columns 1 to 4			
	Computed for track cores	Cone method	Vibrator method	Computed for recompressed mixture	Computed for track cores	Cone method	Vibrator method	Computed for recompressed mixture
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	31.0	33.6	30.0	28.4	64.2	57.1	67.7	72.8
2	31.7	32.7	29.3	27.4	66.8	63.9	75.0	81.2
3	31.0	32.6	29.2	28.1	76.8	71.2	83.7	88.4
4	31.4	32.9	27.5	28.4	80.3	74.8	96.8	92.6
5	31.9	32.3	28.1	29.7	87.1	85.5	104.8	96.9
6	26.3	27.3	21.0	21.2	61.6	58.2	82.4	81.4
7	28.3	27.2	21.8	21.5	61.5	65.0	87.2	88.4
8	26.0	25.3	20.9	21.3	71.5	74.3	95.0	92.9
9	26.8	26.6	22.5	24.5	86.9	88.0	109.7	97.8
10	27.6	26.3	22.2	25.7	92.4	98.8	122.7	101.7
11	30.7	32.2	28.6	26.6	51.2	47.8	56.8	62.6
12	29.3	32.8	28.8	25.7	68.3	57.9	70.3	81.6
13	27.7	32.4	28.2	25.1	76.5	61.1	74.5	87.7
14	31.3	32.2	28.9	26.4	72.2	69.6	81.5	92.1
15	28.1	32.3	27.5	27.0	91.4	74.8	94.0	96.1
16	32.1	36.0	30.8	28.7	57.3	48.1	61.1	67.4
17	32.9	35.8	30.2	27.9	56.2	49.5	63.9	71.3
18	27.5	31.6	27.0	24.4	75.7	62.7	78.8	88.9
19	26.3	28.9	24.0	24.3	87.8	77.5	99.0	97.6
20	31.2	33.2	28.4	26.2	58.9	55.7	70.6	78.4
21	29.4	31.9	28.8	25.9	68.3	60.5	70.2	81.2
22	29.1	30.7	28.8	25.2	67.4	62.2	68.1	80.0
23	30.4	31.8	28.0	25.7	68.4	64.2	77.0	86.7
24	30.5	32.6	28.1	26.0	65.2	59.2	74.2	81.8
25	31.8	31.7	28.0	26.7	60.7	61.5	73.2	78.1
26	34.5	31.8	27.0	25.0	54.8	62.3	78.2	86.7
27	34.9	31.5	27.7	25.0	57.3	66.6	80.3	92.0
28	33.9	31.3	28.0	25.1	54.6	62.3	72.3	84.1

The cone method of determination gave aggregate voids which were generally higher than actually existed in the pavement. Some of the coarse-sand mixtures and some of those laid with the blown asphalts were exceptions. This reversal of the general tendency may indicate the relative incompressibility of these mixtures, although, in the case of the coarse-sand mixtures at least, it is more likely that they did not receive sufficient rolling.

TESTS SHOW A RELATION BETWEEN AGGREGATE VOIDS, PERCENTAGE OF BITUMEN AND STABILITY

The percentages of voids as determined by the various methods show the influence of gradation. The coarse group is characteristically low in voids. The variable filler series illustrates how aggregate density may be increased by the addition of dust. None of the laboratory methods uniformly reproduces the condition attained in the field.

Figure 9 shows the percentages to which the aggregate voids would be filled with bitumen were the several mixtures compressed to the degree corresponding to the aggregate voids determined by the various methods and shown in Figure 8. The data from which this chart is derived are given in Table 6. It appears that the fine-sand mixtures, as compacted in the test sections remain stable with approximately 77 percent of the aggregate voids filled with bitumen. The coarse-sand group seemed able to carry bitumen up to about 75 percent of the aggregate voids. Due probably to variations in sand supply, the behavior of the medium-sand mixtures was somewhat more erratic, due probably to variations in sand supply. Possibly a percentage of bitumen equivalent to 70 percent of the aggregate voids is about the maximum for stable mixtures and this maximum is dependent on the presence of approximately 12 or 13 percent of material passing the No. 200 sieve. There are indications that higher dust contents, such as in mixtures 13 and 18, which

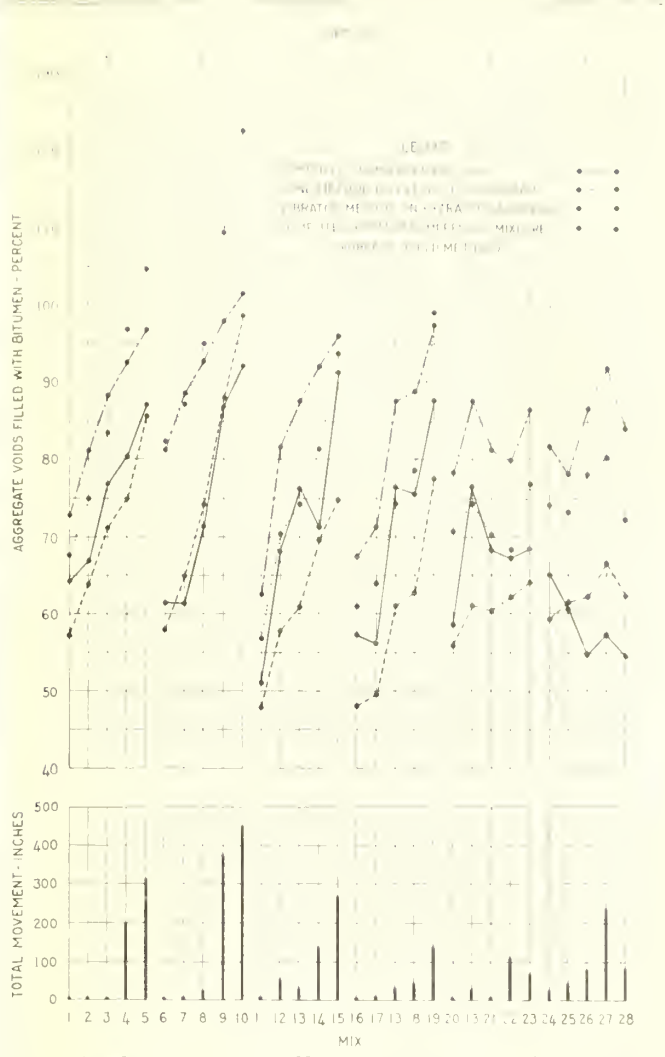


FIGURE 9.—PERCENTAGE OF VOIDS IN AGGREGATE FILLED WITH BITUMEN WHEN MIXTURES ARE COMPRESSED TO DENSITIES CORRESPONDING TO AGGREGATE VOIDS SHOWN IN FIGURE 8.

decrease the size as well as the volume of aggregate voids, will permit a somewhat greater percentage of voids to be filled with bitumen.

If other methods of void determination are used the results may be different and, the ratio of voids to volume of bitumen will be changed. If voids in the aggregate are taken to be those determined by the cone method, the degree to which the voids may be safely filled will be lower than that given in the preceding paragraph. Conversely, a greater percentage may be used on the basis of the lower voids determined by the vibrator method or the Hubbard-Field compression mixture method. Probably the greatest possible density of mixture is attained by the latter method and it is interesting to note that only the extremely low-void aggregates of the coarse-sand series carried, with satisfactory results, an amount of bitumen greater than 90 percent of the aggregate voids as computed by the Hubbard-Field method.

STABILITY OF TEST SECTIONS COMPARED WITH RESULTS OF LABORATORY STABILITY TESTS

Table 7 and figure 10 show comparisons of the observed field stabilities, the stabilities as determined by the Hubbard-Field method upon recompressed specimens, and approximations of Hubbard-Field stabilities of the mixtures at the densities of the sections as constructed. These approximations were made as in-

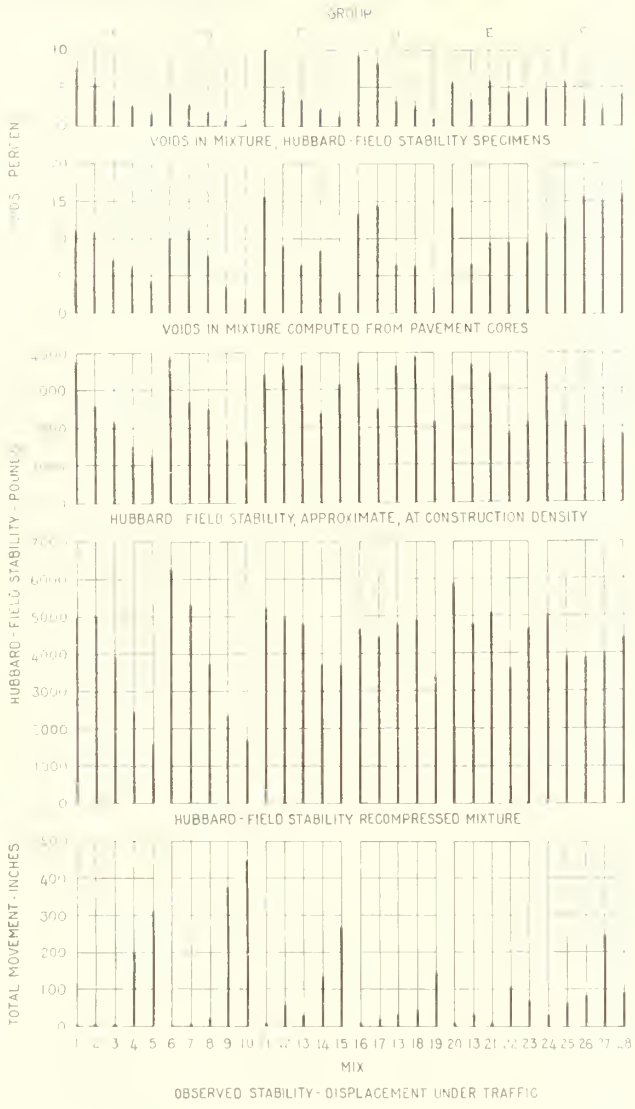


FIGURE 10.—COMPARISON OF STABILITY AND VOIDS OF SHEET ASPHALT MIXTURES.

dicated in figure 11 by interpolating for the determined void content of pavement cores between results obtained in the Hubbard-Field test upon recompressed laboratory specimens and specimens taken from the upper 1 inch of removed pavement slabs. There is excellent agreement between displacement of test sections and the results of stability tests in the laboratory for the fine-sand and coarse-sand mixtures. The results indicate that a Hubbard-Field stability of over 3,000 pounds for laboratory compressed specimens is necessary to withstand the type of traffic to which these mixtures were subjected. At the densities to which the mixtures were compressed in the pavement a stability of about 2,000 is required.

The medium-sand mixtures gave decidedly less consistent agreement in results. All mixtures with a stability of 4,500 or more for laboratory compressed specimens gave reasonably good service. It will be noted that decidedly inferior mixtures such as numbers 14 and 15 have stabilities in laboratory tests approximately equal to that of the satisfactory mixture number 8. The soft-blown-asphalt mixtures, numbers 26, 27, and 28, also gave fairly high stability in laboratory tests. These mixtures were poorly

TABLE 7.—*Comparison of stability and voids in sheet asphalt mixtures*

Mixture or section number	Stability			Voids in mixture		Rank according to—		
	Total displacement of 25 screws[1]	Hubbard-Field test on recompressed specimens[2]	Hubbard-Field test on specimens at approximately construction density	Computed for cores taken from pavement	Computed for recompressed specimens for Hubbard-Field stability test.	Displacement by traffic	Results of Hubbard-Field test on recompressed specimens	Results of Hubbard-Field test on specimens at approximately construction density
	Inches	*Pounds*	*Pounds*	*Percent*	*Percent*			
1	1.4	5,008	3,800	11.0	7.7	1	9	3
2	4.0	5,019	2,600	10.7	5.2	5	8	13
3	2.7	3,986	2,200	7.2	3.2	3	17	17
4	199.8	2,445	1,500	6.2	2.2	23	25	26
5	311.2	1,638	1,250	4.0	1.1	26	28	28
6	1.6	6,269	3,850	10.2	4.0	2	1	2
7	8.3	5,372	2,700	11.0	2.6	9	3	12
8	23.4	3,741	2,500	7.4	1.6	11	21	15
9	379.0	2,355	1,700	3.5	.6	27	26	24
10	448.9	1,697	1,650	2.1	.1	28	27	27
11	6.7	5,244	3,450	15.1	10.0	7	4	7
12	56.1	5,044	3,700	9.3	4.8	16	7	6
13	32.8	4,824	3,700	6.5	3.1	13	11	5
14	137.6	3,723	2,400	8.7	2.1	21	22	16
15	267.4	3,759	3,200	2.4	1.0	25	20	11
16	7.1	4,687	3,750	13.6	9.4	8	12	4
17	15.6	4,466	2,525	14.4	8.0	10	15	14
18	32.8	4,824	3,700	6.5	3.1	13	11	5
19	46.9	4,908	3,900	6.5	2.7	14	10	1
20	140.8	3,376	2,175	3.1	.7	22	24	19
13	2.8	5,849	3,400	13.2	5.7	4	2	8
21	32.8	4,824	3,700	6.5	3.1	13	11	5
22	5.1	5,136	3,425	9.4	5.5	6	5	10
23	109.0	3,619	1,875	9.6	4.6	20	23	22
24	71.1	4,676	2,175	9.5	3.5	17	13	20
25	28.1	5,047	3,400	10.6	4.8	12	6	9
26	48.4	3,955	2,175	12.8	5.9	15	18	18
27	82.9	3,936	2,000	15.6	3.3	18	19	21
28	244.2	4,050	1,675	14.9	2.0	24	16	25
	87.0	4,472	1,800	15.9	3.9	19	14	23

[1] Averaged from movement of 50 screws in two lines. In some cases results are approximate as a result of disintegration.
[2] Average of 3 specimens.

compacted in the field test and their estimated stability based on field voids is more in line with their field performance.

The voids in the mixtures (table 7 and fig. 10) both as laid and after laboratory recompression, are of interest in connection with the stabilities. In each group of mixtures, the highest voids of recompressed specimens are in mixtures having the greatest stabilities both in the Hubbard-Field test and under traffic.

Those mixtures which because of high bitumen content readily compacted to produce pavements having the lowest voids proved the least stable under traffic even though, as in the case of mixtures 14, 15, and 27, the Hubbard-Field stability test indicated good strengh. In general, it may be said that those mixtures, which, because of the high bitumen content were compressible by the indicated method to 2 percent or less of voids, were virtually certain to displace under the traffic used in these tests.

Mixture 8 may appear to be an exception to this statement, but it will be remembered that the aggregate of this group had low voids and the lower voids of the compressed mixtures do not represent greater density obtained by dangerously increased bitumen content.

The blown-asphalt group presents another phase of the density and stability relationship. As a result of inadequate rolling the voids in the pavement were high. Instead of behaving as did high-void mixtures in other groups, mixtures 26 and 28 were relatively unstable, reflecting to some extent perhaps the softer

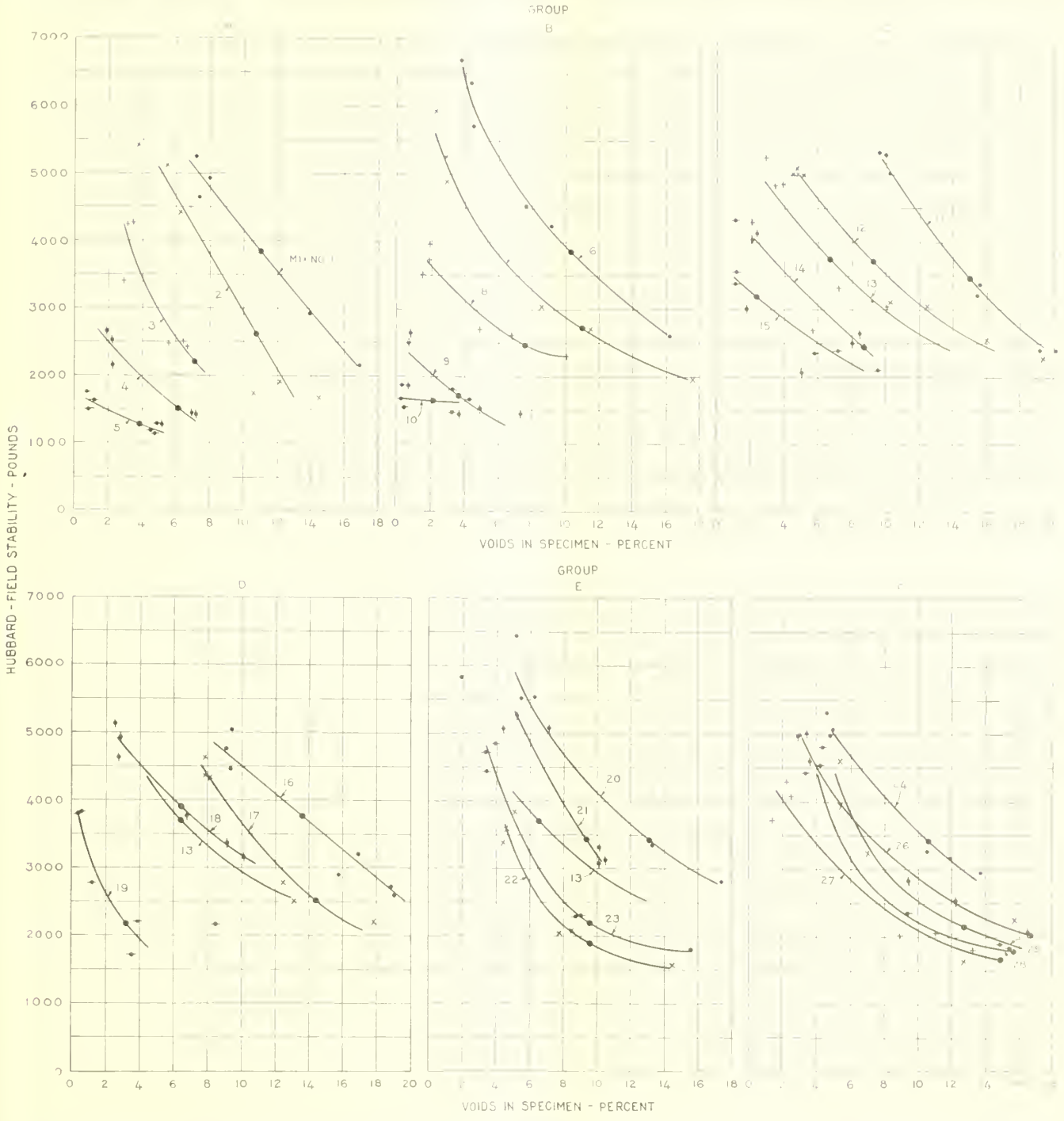


FIGURE 11.—CURVES SHOWING RELATION BETWEEN VOIDS IN SPECIMEN AND HUBBARD-FIELD STABILITY AS DETERMINED ON RECOMPRESSED SPECIMENS AND ON SPECIMENS TAKEN FROM TOP INCH OF PAVEMENT. THE LARGE DOT ON EACH CURVE CORRESPONDS TO THE PERCENTAGE OF VOIDS IN THE PAVEMENT AS CONSTRUCTED. DIFFERENT SYMBOLS ARE USED TO REPRESENT POINTS PERTAINING TO DIFFERENT MIXTURES.

bitumen which they contained, but more probably due to their susceptibility to compression to approximately the danger line of voids for their respective aggregates. Mixture 27 had a harder bitumen than mixture 28 and high voids as laid in the pavement. This mixture proved very unstable in laboratory tests after being compressed to 2.0 percent voids.

In this discussion of asphalt mixtures only stability or resistance to displacement has been considered. In reviewing this work and the results, it should be remembered that unusually severe conditions of traffic were imposed. The smooth base offered no keying action and the test was conducted during the hottest periods of the year when faults in proportioning are most likely to be developed.

ASPHALTIC CONCRETE MIXTURES DISCUSSED

In the earlier stability experiments with asphaltic concrete, those mixtures which contained approximately equal parts of coarse aggregate (retained on the no. 10 sieve) and fine aggregate (passing the no. 10 sieve) were very resistant to traffic displacement. Five asphaltic concrete sections of the second series of tests were patterned after these highly successful mixtures. All

were planned to be of the same aggregate composition with the bitumen content as the only intended variable.

A clean hard limestone was used as the coarse aggregate. The sand and filler were the same as used in all of the sheet-asphalt sections except mixtures 1 to 10 inclusive. The asphalt cement, the analysis of which is given in table 4, had a penetration of 42. At the plant mixture 29 appeared to be extremely lean. Mixture 32 was so rich that it flowed freely and formed a uniform layer over the bottom of the truck, giving rise to the suspicion that it would prove unstable under traffic. Instead of using a still higher percentage of bitumen in mixture 33 as scheduled, a bitumen content between that of mixtures 31 and 32 was adopted.

Table 8 shows the plant formulas with which the asphaltic concretes were prepared, the analyses of large samples taken at the paving plant, their stabilities in the pavement as determined by the displacement of the reference screws, and laboratory determinations of stability with the roller stability machine.[6] No asphaltic-concrete section was displaced to an extreme degree. The maximum movement of a single reference screw in section 33, the least stable of the group, was 3.6 inches.

Table 9 shows the theoretical and actual specific gravities of the several mixtures, the calculated voids in the pavements and in the aggregates, and the extent to which the voids in the aggregates were filled with bitumen. It was recognized that small plugs as punched from the sheet asphalt sections would not be suitable for determining the density of the coarse-graded mixtures as laid in the pavement. Therefore, after the entire test was completed, a number of slabs, approximately 24 inches square, were chopped from each section. In general, eight slabs were taken from each section from areas over which the traffic had not passed, and the average of their specific gravities is regarded as the original density of the pavement section. Two slabs were taken from the wheel-track area and their average densities gives an indication of the increased compression generally resulting from the 64,000 passages of the truck.

The photographs of the asphaltic-concrete sections at the completion of the test, and the data in table 8 show that no mixture was displaced to an extreme degree, despite a variation in bitumen content of more than 50 percent between some of the mixtures. It is true that mixtures 32 and 33 were relatively unstable, reflecting the effect of the high ratio of volume of bitumen to volume of voids in the aggregate. Traffic on these sections during high summer temperatures produced a virtual absence of voids in the area under traffic. That a greater degree of instability did not develop, comparable with the complete failure of the richest sheet asphalt mixtures with voids practically filled with bitumen, must be due chiefly to the inherent rigidity of the interlocked particles of coarse aggregate.

RESULTS OF ROLLER TESTS IN AGREEMENT WITH PERFORMANCE OF MIXTURES IN FIELD TESTS

Specimens were cut from the large field samples and tested in the laboratory on the roller stability machine.[7] This machine was designed to reproduce the action of

[6] Researches on Bituminous Paving Mixtures, Public Roads, vol. 7, no. 10, December 1926

[7] Researches on Bituminous Paving Mixtures, Public Roads, vol. 7, no. 10, December 1926.

TABLE 8.—*Composition, analyses, displacement and roller stability values of asphaltic-concrete mixtures of second series of stability tests*

Section or mix no.	Plant or mixing formula, percent by weight				Analyses of plant sample, passing and retained on next smaller size, percentages by weight														Displacement		Roller stability, number of passages
	Bitumen	Dust	Sand	Stone	Bitumen	1¼ inches	1 inch	¾ inch	½ inch	¼ inch	No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200	Maximum movement of single screw, inches	Movement of 25 screws, inches[1]	
29	5.1	3.5	43.7	47.7	4.8	0.0	14.6	17.0	8.6	3.0	3.3	4.3	3.4	8.2	12.0	6.8	8.5	5.5	0.2	0.8	355
30	6.0	3.4	43.2	47.4	5.8	4.7	12.7	16.1	10.4	4.8	3.0	4.0	3.0	6.6	9.2	5.2	8.6	5.9	.1	.5	297
31	7.1	3.4	42.8	46.7	7.1	3.6	9.6	11.7	10.9	5.7	3.7	4.5	3.4	7.2	10.5	6.2	9.1	6.8	1.2	5.1	244
32	8.1	3.3	42.3	46.3	7.6	4.3	10.0	13.9	10.9	5.6	4.5	4.6	3.3	6.9	8.7	6.1	7.6	6.0	2.9	21.1	88
33	7.6	3.4	42.5	46.5	7.3	2.7	11.8	16.1	9.1	7.0	4.8	5.0	3.3	6.5	8.4	4.8	6.5	6.7	3.3	21.5	99
Average gradation, coarse aggregate						6.8	26.1	33.2	22.3	11.6											
Average gradation fine aggregate											8.0	9.3	6.7	14.6	20.1	11.9	16.6	12.8			

[1] Averaged from movement of 50 screws in 2 lines.

TABLE 9.—*Density and voids of asphaltic-concrete mixtures in traveled and untraveled areas*

Mix no.	Theoretical maximum specific gravity		Average specific gravity of pavement slabs		Voids in compressed mixtures				Voids in aggregate				Percentage of aggregate voids filled with bitumen			
	Based upon extraction	Based upon plant formula	No traffic	Subjected to traffic	Based on extraction composition		Based on plant formula composition		Based on extraction composition		Based on plant formula composition		Based on extraction composition		Based on plant formula composition	
					No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic	No traffic	Subjected to traffic
					Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
29	2.490	2.480	2.309	2.277	7.3	8.6	6.9	8.2	17.8	19.0	17.9	19.2	59.0	54.8	62.6	59.2
30	2.456	2.450	2.332	2.404	5.1	2.1	4.8	1.9	18.0	15.4	18.1	15.6	71.6	86.3	73.4	87.8
31	2.417	2.417	2.329	2.403	3.6	.6	3.6	.6	19.3	16.8	19.3	16.8	81.3	96.6	81.3	96.6
32	2.393	2.379	2.378	2.390	.6	.1	0.04	.0	17.8	17.4	18.3	18.4	96.6	99.4	99.2	100.+
33	2.410	2.400	2.387	2.395	1.0	.6	.5	.2	17.6	17.2	17.8	17.5	94.3	96.6	97.2	98.9

traffic over a pavement surface. It consists essentially of a series of 11 steel cylinders or rollers, 4 inches in diameter and 3 inches long, mounted between and near the peripheries of two confining steel disks, which in turn are rotated by a motor. Beneath the rollers is a tank of water maintained at a temperature of 60° C. by an electric heater, and in which the specimen to be tested is placed. This bath is supported on 4 cams which rotate, raising and lowering the bath to compensate for the vertical component of the arc described by the roller in passing over the specimen, thereby eliminating impact.

The specimen is supported in a testing mold having one end and the top surface open. Rotation of the rollers is induced as they pass over the top surface of the specimen, tending to deform it longitudinally through the open end of the mold. The deformation is measured with an Ames dial. A trip indicator records the number of rolls passing over the specimen. The total weight of the bank of rollers is 450 pounds, giving a load of 150 pounds per inch width of roller as they pass over the specimen.

To prevent upward deformation at the sides of the specimen, a small section of angle-iron is clamped over its edges, extending ½ inch over the top at either side, leaving a 3-inch open surface over which the rollers pass. The specimen to be tested is brought to a temperature of 60° C. and is placed in the bath. The revolving rollers are lowered in contact with its surface. Rolling is continued until 0.3 inches longitudinal deformation has taken place. The number of rollers passing over the specimen in producing this deformation is read from the trip indicator and recorded as the roller stability value of the specimen. Results of this test were in general agreement with the service behavior of the asphaltic-concrete mixtures, those composing sections 32 and 33 deforming far more readily than did those which had more successfully resisted the truck traffic.

OBSERVATIONS ON SHEET ASPHALT MIXTURES

1. Equally stable sheet asphalt surface mixtures were laid with all three of the sands used. The fine and coarse sands were considerably outside the gradation limits of the usual specifications.

2. The effects of the individual characteristics of the constituent materials are so interrelated that in designing a sheet asphalt mixture it is necessary to consider the properties of various combinations of the materials.

3. The voids in the aggregate are of decided significance with respect to the amount of asphalt which the mixture can carry satisfactorily.

4. Aggregate voids may be determined by a number of methods but all of the several methods used in this investigation gave different results. The most satisfactory method was that of determination on compressed mixture specimens, prepared as for the Hubbard Field stability test. In general, this method gave lower voids than any other method.

5. No mixture having the voids completely filled with bitumen remained stable under the traffic imposed in these tests. In general, the maximum percentage of bitumen carried by stable mixtures amounted to between 85 percent and 90 percent of the aggregate voids as determined for Hubbard-Field stability specimens.

6. There are indications that mixtures containing aggregates with low percentages of voids may carry amounts of bitumen representing somewhat higher percentages of the aggregate voids. No mixture studied gave satisfactory results where the aggregate voids were filled in excess of 93 percent.

7. Stable mixtures resulted from the use of asphalts of 35 to 55 penetration. Softer steam-refined asphalts of 63 and 72 penetration gave somewhat more plastic mixtures, although but one section was laid with each of these consistencies.

8. The few sections in which blown asphalt was used were unsatisfactory due to the deterioration of the asphalt resulting in either actual or incipient disintegration of the pavement mixtures.

9. During construction the mixtures probably were not compressed to the maximum possible degree, although in general those which showed the highest voids in the pavement were also the least compressible in the laboratory. From the standpoint of stability, low voids in the compressed mixture appear to be undesirable if void reduction is accomplished by the addition of bitumen beyond the limits defined in 5 and 6. In each group of mixtures, except possibly the blown asphalt series, those with the higher percentages of voids were the more stable. This is further evidence of the importance of voids being filled with asphalt to the proper degree.

10. Since field compression is relatively nonuniform, the relative compressibility of mixtures can best be determined in the laboratory by standard test methods.

11. In general, service behavior of the mixtures was approximately proportional to stability values determined in the laboratory by the Hubbard-Field method. There were a few instances in which laboratory tests on mixtures found to be plastic in field tests indicated stabilities about as high as for mixtures found to be stable under traffic. In these cases, however, the voids in specimens compressed in the laboratory were dangerously low.

12. Air temperature prevailing during the daily traffic periods exerted a strong influence upon the behavior of the mixtures. With average air temperatures lower than 70° F. all sections were stable. At increased temperatures instability first became evident in those sections which were expected to prove least stable.

OBSERVATIONS ON ASPHALTIC-CONCRETE MIXTURES

14. Asphaltic concrete mixtures of dense graded aggregates appear well able to resist the heavy concentrated traffic imposed in these tests. The mixtures of the second series were outstandingly successful. They contained approximately equal parts of coarse and fine aggregate.

15. The limits of range in bitumen content which can be used without danger of extreme displacement are much less critical with the dense-graded asphaltic concretes than with sheet asphalt.

16. Although it is recognized that asphaltic concretes containing well over 60 percent of coarse aggregate have been successfully used in many localities, such mixtures behaved poorly in the first series of stability tests. An unintentional enrichment in bitumen caused by penetration of the seal coat was probably chiefly responsible for the failures which occurred, but it is believed that the more densely graded mixtures, successfully used in both series of tests, are inherently

(Continued on p. 218)

RELATIVE VISCOSITIES OF LIQUID ASPHALTIC ROAD MATERIALS AT VARIOUS TEST TEMPERATURES

By J. T. PAULS, Senior Highway Engineer, and R. H. LEWIS, Associate Chemist, Division of Tests, Bureau of Public Roads.

A REVIEW of the 1931 State specifications, submitted to the Bureau of Public Roads in connection with the cooperative work on the simplification of tests for liquid asphaltic materials, showed that there was considerable difference of opinion as to the temperatures at which viscosity determinations should be made. The following tabulation shows the various temperatures designated for the test and the number of specifications in which each temperature was designated

Designated temperature (°F.)	Number of specifications
77	39
104	12
122	87
140	16
194	2
210	1
212	7
302	1

The consistency of material for hot application was controlled not only by viscosities at the higher temperatures (194° F., 212° F., and 302° F.), but also by the float test at 90° or 122° F. and, in some cases, by both a viscosity and a float test.

WIDE DIFFERENCES OF OPINION EXPRESSED AS TO TEMPERATURE FOR VISCOSITY TESTS

At regional meetings of State testing engineers for consideration of the various tests, the Saybolt-Furol method of viscosity determination was adopted. The temperatures recommended by most of these groups for viscosity tests were 77° F., 122° F., and 210° F. There were, however, some who favored the float test at 122° F., as a single measure of consistency instead of the Furol viscosity test at 210° F.

There was a fairly general agreement on the choice of 122° F. as a test temperature with the following exceptions:

(a) Two States voted for the use of 104° F. as a test temperature instead of 122° F.

(b) Eight States of the Mississippi Valley region, although they did not vote for the Furol viscosity determinations at temperatures other than 77° F., 122° F., and 210° F., expressed the belief that some temperature between 122° F. and 210° F. was desirable.

(c) Seven States voted for the use of 140° F. as a control temperature for viscosities in addition to the three temperatures generally accepted.

The 1932 State specifications also show a wide difference of opinion as to the proper temperature to be used for viscosity determinations. In addition to the temperatures already listed, there have been specified during 1932 two others—180° F. and 200° F. It is apparent that there is considerable doubt as to the sufficiency of 122° F. as the single intermediate temperature; and there is a question as to whether a high-temperature viscosity test for the control of materials for hot application could not be discarded for some other consistency test.

The 1931 specifications gave the following maximum viscosity limits as determined with the Saybolt-Furol apparatus:

Temperature (°F.)	Maximum time, seconds
77	480
104	436
122	1,200
140	260
194	440
210	180
212	320

One of the important advantages of the Furol instrument over the Engler is the shorter time required to make a viscosity determination. This time advantage should be retained by the selection of test temperatures such as will insure accurate determinations in a reasonable time. In the above table the maximum time for the test at 122° F. is much greater than for the other temperatures. The maximum time of 1,200 seconds, or 20 minutes, shows the extent to which the time advantage of the Furol instrument may be lost.

VISCOSITY TESTS AT 210° F. WILL NOT FURNISH SHARP DIFFERENTIATION BETWEEN MATERIALS FOR RETREAD AND ROAD-MIX CONSTRUCTION

While there were only 12 specifications in 1931 having maximum viscosity limits of over 500 seconds, Furol at 122° F., there is nevertheless a trend toward the use of more viscous materials. Materials, considerably thinner than those for hot application, are used extensively in retread and oiled road-mix construction. The viscosities of these asphaltic products are very high at 122° F. The next higher temperature recommended at the regional meetings was 210° F. Tests were run at this temperature on a selected number of road oils and kerosene cut-back asphalts to determine the suitability of the temperature for control of such materials. Float tests at 122° F. were also made on the same samples.

The Furol viscosity at 122° F. for liquid asphaltic road materials of the slow-curing type may be compared with the Furol viscosity at 210° F. and the float test results at 122° F. from the data given in table 1 and plotted in figure 1. The materials tested were from many different fields and were, no doubt, produced by quite different methods of processing. No satisfactory curve could be drawn in figure 1 to show the relationship for the different consistency values, although a definite trend is indicated.

The same relationships for kerosene cut-backs are given in table 2 and figure 2. Although the sources of these cut-backs are all different, a fairly satisfactory curve can be drawn to show the relationship between viscosities at 122° F. and viscosities at 210° F. and between viscosities at 122° F. and float tests at 122° F.

These relationships may be expressed approximately as follows:

$$C = 0.03\,B + 18$$
$$D = 0.02\,B + 10$$

Where B = Furol viscosity at 122° F.
C = Furol viscosity at 210° F. and
D = Float test at 122° F.

The narrow range in viscosities at 210° F. and in float-test results at 122° F. for materials which had a wide range of viscosity at 122° F. indicates that these tests (viscosity at 210° F. and float test at 122° F.) will not furnish sharp differentiation between various consistencies.

140° F. SUITABLE TEMPERATURE FOR VISCOSITY TESTS FOR MATERIALS HAVING HIGH VISCOSITY AT 122° F.

To determine a suitable temperature for viscosity tests on materials for retread and road mix construction, five selected kerosene cut-backs, having appreciable differences in viscosity at 122° F., were tested for viscosity at the following temperatures: 122° F., 140° F. 158° F., 176° F., 194° F., and 210° F. The results of these determinations are given in table 3 and are shown graphically in figure 3.

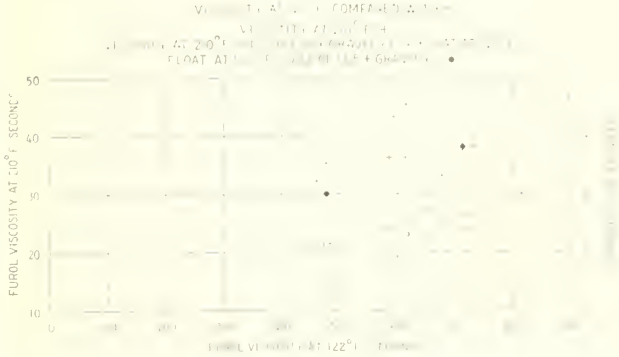


FIGURE 1.—VISCOSITY OF SLOW-CURING LIQUID ASPHALTS AT 122° F. COMPARED WITH VISCOSITY AT 210° F. AND FLOAT-TEST RESULTS AT 122° F.

TABLE 1.—*Furol viscosities of liquid asphaltic road materials of the slow-curing type at 122° F., 140° F., and 210° F., and float tests at 122° F.*

Sample no.	Producer	Specific gravity	122° F.		Viscosity at 140° F.	Viscosity at 210° F.	Float test at 122° F.	Results of other operators at 122° F.	
			Viscosity	Time of drip				Maximum viscosity	Minimum viscosity
			Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*
34,453	Utah Refining Co.	1.079	68		36				
35,364	Shell, Wood River	.942	68		47			68	64
34,263	White Eagle	1.066	94		48			105	90
35,100	Shell, East Chicago	1.001	94		53			91	89
35,077	Shell, Martinez	.951	111		63			111	111
36,303	Standard Oil Co. of California, Bakersfield	.967	151		80			[1] 150	150
36,106	Standard Oil Co. of California, Richmond	.952	178		94			[1] 174	174
35,367	Shell, Wood River	.955	199		106			194	185
35,226	Gilmore, Los Angeles	.967	201		104			198	198
35,103	Shell, East Chicago	1.003	205		110			196	192
36,310	Standard Oil Co. of California, Bakersfield	.972	274		134			[1] 280	280
35,080	Shell, Martinez	.963	281		142			300	261
35,157	White Eagle	.976	294		152			308	245
34,454	Utah Refining Co.	1.101	297		123				
35,225	Gilmore, Los Angeles	.972	300		148			311	255
36,089	Standard Oil Co. of California, Richmond	.960	300		149			300	287
34,322	Lincoln Oil Co.	1.082	320		137			345	292
36,091	Standard Oil Co. of California, Richmond	.966	458		220	32	21	500	444
34,323	Lincoln Oil Co.	1.088	478		188	21	30	496	444
36,360	Berry	.981	478	405	243	35	15		
34,467	Union, Oleum		586	440	268	36	24	580	530
36,946	MacMillan	.979	596	480	301	43	19	566	564
35,081	Shell, Martinez	.972	613	[2]	288	36	23	630	616
35,221	Texas, Port Neches	.975	616	540	300	45	23	627	567
36,300	Standard Oil Co. of California, Bakersfield	.981	677	400	295	33	25	720	680
35,155	Standard Oil Co. of Indiana, Wood River	1.087	713	560	271	26	38	707	660
36,211	Standard Oil Co. of California, El Segundo		716	480	325	38	24	[1] 798	798
35,228	Gilmore, Los Angeles	.979	735	[2]	333	38	26	[1] 706	706
36,214	Standard Oil Co. of California, El Segundo		737	450	333	39	24	760	730
35,183	Standard Oil Co. of Indiana, Greybull	.982	808	540	384	45	30	831	790
34,279	White Eagle		838	600	392	48	30	852	805
36,426	Texas, Cody	1.003	898	600	410	46	33	908	850

[1] Only 1 determination.

[2] Not recorded.

TABLE 2.—*Furol viscosities of kerosene cut-backs at 122° F., 140° F., and 210° F., and float test at 122° F.*

Sample no.	Producer	Location	122° F.		140° F.		Viscosity at 210° F.	Float test at 122° F.	Results of other operators at 122° F.	
			Viscosity	Time of drip	Viscosity	Time of drip			Maximum viscosity	Minimum viscosity
			Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*
36,306	Standard Oil Co. of California	Bakersfield	488		238		33	20	550	530
36,095	do	Richmond	787	540	366		43	25	820	750
36,309	do	Bakersfield	999	550	432	410	48	30	1,080	991
35,114	Union Oil Co.	Oleum	1,020	500	445	420	49	33	964	806
35,113	do	do	1,023	600	419	420	47	30	1,105	950
36,097	Standard Oil Co. of California	Richmond	1,120	600	486	425	54	30	1,175	1,100
35,112	Union Oil Co.	Oleum	1,247	600	542	435	56	33	1,238	1,015
36,524	Associated	California	1,393	700	567	450	55	36	1,450	1,249
36,099	Standard Oil Co. of California	Richmond	1,400	600	600	435	62	35		
36,094	do	do	1,411	630	559	450	59	34	1,542	1,400

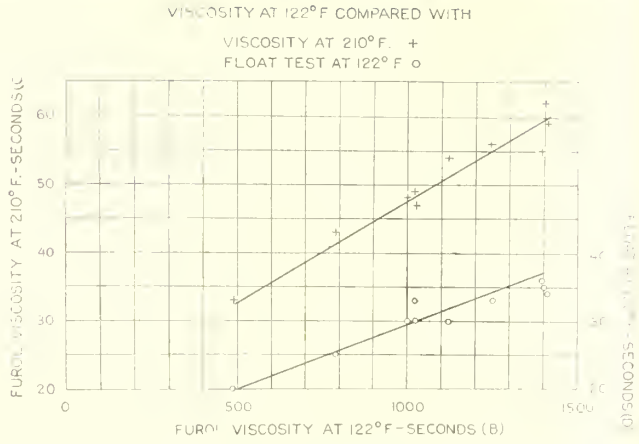

VISCOSITY AT 122°F COMPARED WITH
VISCOSITY AT 210°F. +
FLOAT TEST AT 122°F o
FUROL VISCOSITY AT 210°F.-SECONDS(C)
FUROL VISCOSITY AT 122°F-SECONDS (B)

FIGURE 2.—VISCOSITY OF KEROSENE CUT-BACKS (MEDIUM-CURING TYPES) AT 122° F. COMPARED WITH VISCOSITY AT 210° F. AND FLOAT-TEST RESULTS AT 122° F.

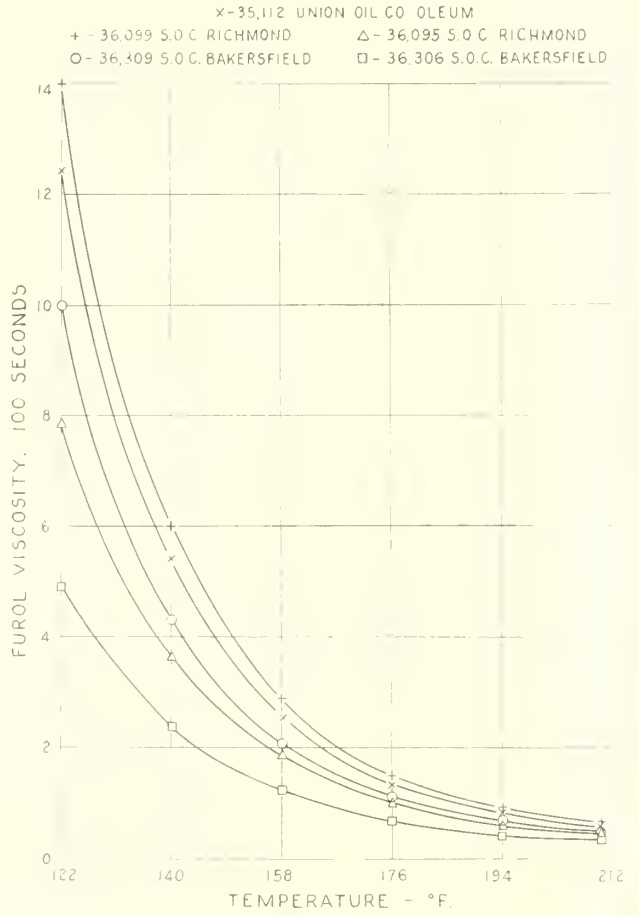

X-35,112 UNION OIL CO OLEUM
+ - 36,099 S.O.C. RICHMOND
△ - 36,095 S.O.C. RICHMOND
O - 36,309 S.O.C. BAKERSFIELD
□ - 36,306 S.O.C. BAKERSFIELD
FUROL VISCOSITY. 100 SECONDS
TEMPERATURE - °F.

FIGURE 3.—TEMPERATURE-VISCOSITY CURVES FOR CALIFORNIA CUT-BACKS (MEDIUM-CURING TYPES).

An 18° increase in temperature from 122° F. to 140° F. decreases the time of the Furol viscosity more than one half even for the least viscous material. A 140° F. temperature has been used by some of the States and is being considered for use by other States, and it was thought desirable to investigate the viscosity values at this temperature of a considerable number of liquid asphaltic road materials of the slow, medium, and rapid-curing types.

The data for viscosities at 140° F. for materials of the slow-curing type are given in table 1, and for materials of the medium-curing type in table 2. Naphtha

TABLE 3.—*Furol viscosities of kerosene cut-backs at various temperatures*

Sample no.	Producer	Viscosity at—					
		122° F	140° F.	158° F.	176° F.	194° F.	210° F.
		Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*	*Sec.*
36,306	Standard Oil Co. of California, Bakersfield	488	238	121	69	44	33
36,095	Standard Oil Co. of California, Richmond	787	366	187	101	62	43
36,309	Standard Oil Co. of California, Bakersfield	999	432	207	111	67	48
35,112	Union Oil Co., Oleum	1,247	542	258	136	81	56
36,099	Standard Oil Co. of California	1,400	600	290	151	88	62

cut-backs (materials of the rapid-curing type) were not used in the preliminary work, but a number of samples of various consistencies from different producers have been tested at 140° F. as well as 122° F. The data on these naphtha cut-backs are given in table 4. The data for liquid asphaltic road materials of the slow-curing type are plotted in figure 4, for kerosene cut-backs in figure 5, and for naphtha cut-backs in figure 6.

In drawing the curve for materials of the slow-curing type in figure 4, it was found that some of the points differed greatly from the general trend of results, and, in checking the test reports on these samples, it wa found that these particular materials all had a hig specific gravity (greater than 1.05), while the gravities of the other materials were all less than 1.003. Tw curves were therefore drawn and the relationship may be expressed approximately as follows:

$$A = 0.45\ B + 15, \text{ for the low gravity group,}$$
$$A = 0.36\ B + 15, \text{ for the high gravity group,}$$

Where A = Furol viscosity at 140° F., and
B = Furol viscosity at 122° F.

The relationship for the kerosene cut-backs, as show in figure 4, may be expressed approximately as follows:

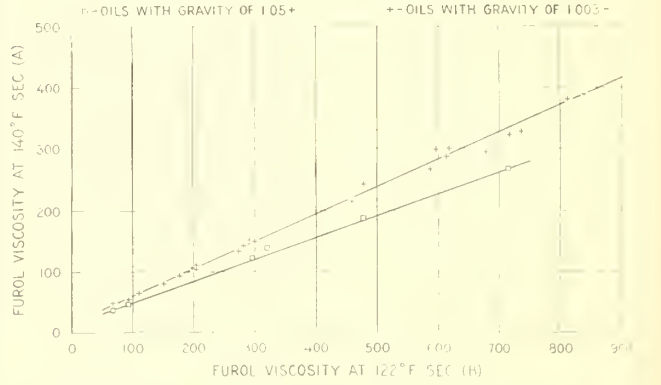


FIGURE 4.—VISCOSITY OF SLOW-CURING LIQUID ASPHALTS AT 122° F. COMPARED WITH VISCOSITY AT 140 F.

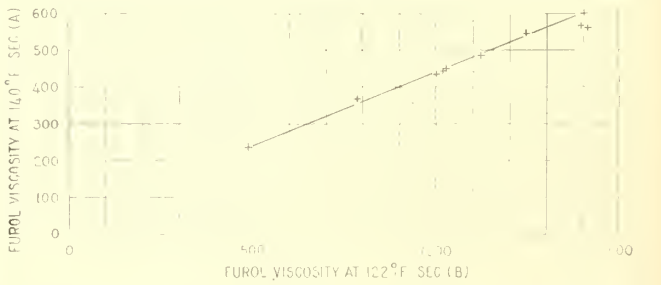


FIGURE 5.—VISCOSITY OF KEROSENE CUT-BACKS (MEDIUM-CURING TYPES) AT 122° F. COMPARED WITH VISCOSITY AT 140° F.

$$A = 0.4\,B + 40$$

Where A = Furol viscosity at 140° F., and
B = Furol viscosity at 122° F.

The relationship for naphtha cut-backs, as shown in figure 6, is approximately as follows:

$$A = 0.4\,B + 60$$

Where A = Furol viscosity at 140° F. and
B = Furol viscosity at 122° F.

TABLE 4.—*Furol viscosities of naphtha cut-backs at 122° F. and 140° F.*

Sample no.	Producer	Viscosity at—		Results of other operators at 122° F.	
		122° F.	140° F.	Minimum	Maximum
		Sec.	*Sec.*	*Sec.*	*Sec.*
35486	Shell, Norco	420	226	426	455
34286	Barber	662	338	651	725
34314	Standard Oil Co. of New Jersey, Baltimore	680	358	729	804
35208	Standard Oil Co. of Louisiana	681	325	685	702
34377	White Eagle	688	322	557	792
35347	Standard Oil Co. of Indiana, Whiting	689	327	676	809
35478	Texas, Port Neches	725	334	577	844
36059	Shell, Norco	730	360	604	875
36058	do	854	432	756	1,060
35207	Standard Oil Co. of Louisiana	919	418	895	914
34317	Standard Oil Co. of New Jersey, Baltimore	919	404	900	936
35474	Texas, Port Neches	960	430	844	1,060

It is believed that a continuous flow through the aperture of the viscosimeter is necessary to obtain an accurate viscosity determination. The control temperature should be so adjusted as to avoid the dripping which occurs when high viscosities are run. In table 1 there is dripping on all viscosities over 478 at 122° F., and when these same samples are run at 140° F. no dripping occurs.

A majority of the kerosene cut-backs listed in table 2 have much higher viscosities than the road oils listed in table 1, and drip at both 122° F. and 140° F., although the difference between the time of drip and the viscosity value is much smaller at 140° F. than at 122° F. The time of drip was not recorded in the naphtha cut-back determinations, but it is probable that dripping did occur on the more viscous materials at both temperatures.

It is evident from a study of the results of "other operators" in viscosity determinations at 122° F. (on same materials tested in other laboratories) that the difference in test values in cases of the heavier materials (tables 1, 2, and 4) are quite large. It is reasonable to suppose that, if accurate temperature control is obtained, the differences in viscosity values obtained by different operators should be materially reduced by the use of 140° F. as a temperature control. The total time for the viscosity determination is greatly shortened, and uncontrollable factors which affect the accuracy of the determinations have considerably less time to influence the flow of oil into the receiver. Since the time of running the test at 140° F. is greatly reduced and the dripping is less, check tests by different operators should be in closer agreement.

Since 140° F. also represents the approximate maximum temperature reached in the upper portion of the road under summer temperature, it is thought that it is a desirable intermediate temperature for the control of the more viscous products which have high Furol viscosities at 122° F.

FIGURE 6.—VISCOSITY OF NAPHTHA CUT-BACKS (RAPID-CURING TYPE) AT 122° F. COMPARED WITH VISCOSITY AT 140° F.

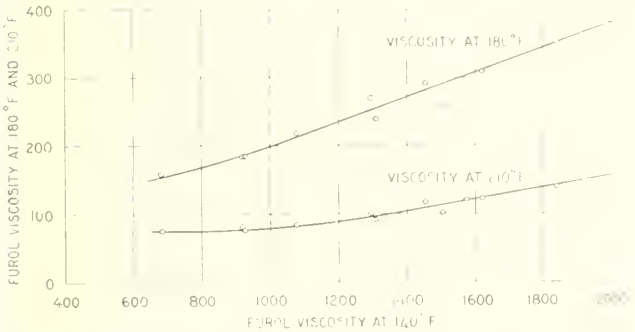


FIGURE 7.—VISCOSITY OF KEROSENE CUT-BACKS (MEDIUM-CURING TYPE) AT 140° F. COMPARED WITH VISCOSITIES AT 180° F. AND 210° F.

Recently very viscous liquid asphaltic road materials of the medium-curing and rapid-curing types have been used as binders in road mixes. Mixtures are prepared in a portable mixing plant and immediately laid and compacted. These materials have exceedingly high viscosities at 140° F. If naphtha has been used as the cutting agent in such products, it is not thought advisable to make the viscosity determination at a temperature above 140° F. because of the possible loss of volatile matter and subsequent stiffening of the material during the test. If kerosene is the solvent used in these cut-backs, a viscosity determination at some temperature higher than 140° F. is not only possible but is desirable because of the shorter time involved in making the test. Several states using material of this grade have designated 180° F. as the control temperature for the viscosity determination.

The Furol viscosities of extremely viscous medium-curing materials at 140° F., 180° F. and 210° F., as well as float tests at 122° F., are given in table 5. Curves showing the relationship existing are plotted in figure 7. It is evident that material having a high viscosity at 140° F. can be satisfactorily controlled by a Furol viscosity at 180° F. and that the use of 210° F. as a viscosity temperature control for this type of material is not necessary.

VISCOSITY TEST AT HIGH TEMPERATURE NOT NEEDED FOR MATERIALS FOR HOT SURFACE TREATMENT

The consistency requirements for materials to be used in hot surface treatments are generally specified by viscosities at one of the higher temperatures or by a float test at 90° F. or 122° F. In many cases the consistency is controlled by both a viscosity at a high temperature and a float test.

The method of applying these hot materials to the road surface is similar to that used in constructing

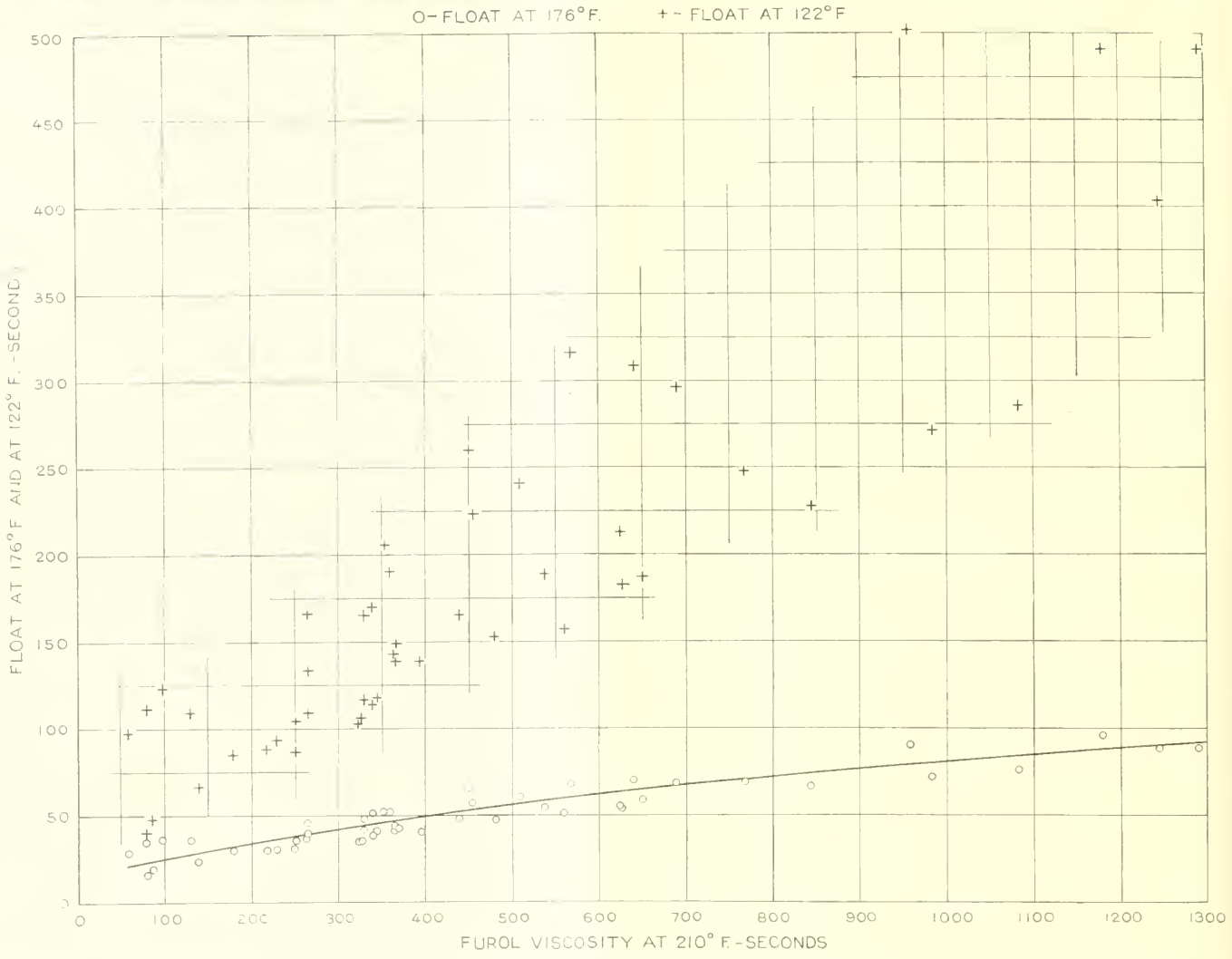


FIGURE 8.—VISCOSITY OF MATERIAL FOR HOT SURFACE TREATMENT AT 210° F. COMPARED WITH FLOAT-TEST VALUES AT 122° F. AND 176° F.

penetration macadam. No viscosity requirement is made in specifying materials for penetration macadam.

It is believed that a viscosity requirement at high temperature for material for hot surface-treatment material can be discarded from specifications and that some test measuring the consistency of the material at a lower temperature can be substituted.

TABLE 5.—*Furol viscosities and results of float tests on kerosene cut-backs at various temperatures*

Sample no.		Producer	Viscosity at—			Float test at 122° F.
Base	Flux		140° F.	180° F.	210° F.	
[illegible]769	36,608	Mexican Petroleum Co. (Baltimore)	1,920	372	148	75.5
			1,840	352	139	70.0
			1,455	290	116	58.0
			682	159	75	40.0
[illegible]326	(1)	White Eagle (Casper)	1,622	309	122	78.0
			1,576	281	121	------
			1,298	270	100	68.5
			920	183	81	55.0
[illegible],120	35,122	Union Oil Co. of California	1,506	256	100	67.0
			1,310	240	90	64.5
			1,077	218	84	59.0
			925	183	76	50.5

(1) Kerosene, no number.

With this idea in mind, a large number of liquid asphaltic road materials suitable for hot surface treatment were tested for Furol viscosities at 210° F., for float at 122° F., 140° F., 158° F., and 176° F., and penetration at 77° F. under variable load and time. The results of this series of tests are given in table 6 and the relationship between Furol at 210° F. and float test at 122° F. and 176° F. are shown in figure 8. The effect of change in temperatures on the float test values for a selected number of samples are shown in figure 9.

It is evident that the float test at 122° F. has no direct relationship to Furol viscosity at 210° F. for the materials tested, although data not included in this report indicate that there is a definite relation between time of float test at 122° F. and viscosities at 210° F. if the materials have been produced from the same base materials and by the same process. If the float tests are made at a higher temperature, the test becomes more nearly a measure of consistency for all types of material than is the case at low temperatures. As is shown in figure 8, a fairly satisfactory curve can be drawn to show the relationship between Furol viscosity at 210° F. and float test results at 176° F.

TABLE 6.—*Furol viscosities at 210° F., float tests at 122° F., 140° F., 158° F., 176° F., and penetrations at 77° F. for slow-curing materials suitable for hot surface treatment*

Sample no.	Producer	Viscosity at 210° F.	Float test at—				Penetrations at 77° F.			
							50 grams		100 grams	
			122° F.	140° F.	158° F.	176° F.	5 sec.	1 sec.	5 sec.	1 sec.
		Sec.	*Sec.*	*Sec.*	*Sec.*	*Sec.*				
35145	Standard Oil Co. of Indiana, Wood River	60	97	55	40	28		249		
35265	Standard Oil Co. of New Jersey, Bayonne	83	39	27	21	16		(1)		
35356	Standard Oil Co. of Indiana, Whiting	83	111	74	48	35		180		
34254	The Texas Co., Bayonne	89	48	32	23	18		(1)		
35196	Standard Oil Co. of New Jersey, Parkersburg	100	123	77	51	36	310+	189		
35353	Standard Oil Co. of Indiana, Whiting	133	108	70	48	35		191		
36044	Jas. B. Berry	143	66	42	32	24		(1)		
35266	Standard Oil Co. of New Jersey, Bayonne	181	85	56	36	30		(1)		
35267	——do	220	88	58	40	30		(1)		
34248	The Texas Co., Bayonne	232	93	60	41	30	300+	(1)		
36343	Shell Oil Co., Norco	253	87	64	42	32		(1)		
35048	Standard Oil Co. of New Jersey, Baltimore	254	104	74	50	35		242		
36307	Standard Oil Co. of California, Bakersfield	266	166	107	68	46	294+	105		
36863	Mexican Petroleum Corporation, Baltimore	267	109	74	50	37		(1)		
36216	Standard Oil Co. of California, El Segundo	267	133	83	57	39	283+	175		
34497	The Texas Co., Port Neches	326	103	71	50	35		(1)		
36187	Atlantic Refining Co., Brunswick, Ga	328	105	76	50	35		266		
36085	Mexican Petroleum Corporation, Destrahan, La	331	117	79	57	41		249		
36093	Standard Oil Co., Richmond	333	165	104	69	48	285+	118		
36759	The Texas Co., Bayonne	342	115	75	54	38		(1)		
35083	Shell Oil Co., Martinez, Calif	342	171	111	75	51	277+	111	303	93
35335	Standard Oil Co. of New York, Riverside	347	118	80	56	40		208		
36311	Standard Oil Co. of California, Bakersfield	355	206	119	82	52	193	83	232	
35115	Union Oil Co., Oleum	362	191	114	76	52	242	115	291+	106
37323	The Texas Co., Bayonne	366	142	92	63	42	255+	149		
35348	Standard Oil Co. of Indiana, Whiting	367	139	89	65	43	315+	263		
35049	Standard Oil Co. of New Jersey, Baltimore	369	149	97	65	42	281+	172		
35013	Standard Oil Co. of Ohio	398	139	92	61	41	290+	169		
35192	Standard Oil Co. of New Jersey, Baltimore	442	166	103	72	48	283+	132		
35082	Shell Oil Co., Martinez, Calif	452	260	140	96	67	157	63	210	
35229	Gilmore Oil Co	456	223	133	87	57	158	79	250	
36702	Mexican Petroleum Corporation, Baltimore	484	153	100	71	47	290+	155		
36092	Standard Oil Co. of California, Richmond	512	242	136	88	61	146	63	194	
36320	Standard Oil Co. of Louisiana	540	189	120	84	55	237	120	295+	186
36344	Shell Oil Co., Norco	563	158	108	77	52	309+	167		
36525	Associated Oil, Avon	570	316	159	105	68	126	52	160	
34496	The Texas Co., Port Neches	627	212	124	86	56	222	136	305+	199
34273	Colonial Beacon Oil Co	628	183	111	82	55	273	144	302+	240
35116	Union Oil Co., Oleum	644	308	159	97	70	127	69	175	
36086	Mexican Petroleum Corporation, Destrahan, La	653	187	115	83	59	266	136	292+	198
35156	White Eagle	693	297	157	102	69	120	60	180	
36319	Standard Oil Co. of Louisiana	770	248	147	99	69	165	92	235	
36345	Shell Oil Co., Norco	846	228	133	93	67	238	120	298+	197
36215	Standard Oil Co. of California, El Segundo	960	501	213	129	90	85	41	121	
36353	Atlantic Refining Co., Brunswick, Ga	985	271	147	100	72	188	95	264	
36087	Mexican Petroleum Corporation, Destrahan, La	1,085	285	165	109	76	159	85	220	
35230	Gilmore Oil Co	1,181	491	221	133	96	97	43	158	
36318	Standard Oil Co. of Louisiana	1,248	403	190	124	88	126	65	169	
34495	The Texas Co., Port Neches	1,293	491	221	131	88	110	61	162	

1 Too soft for test.

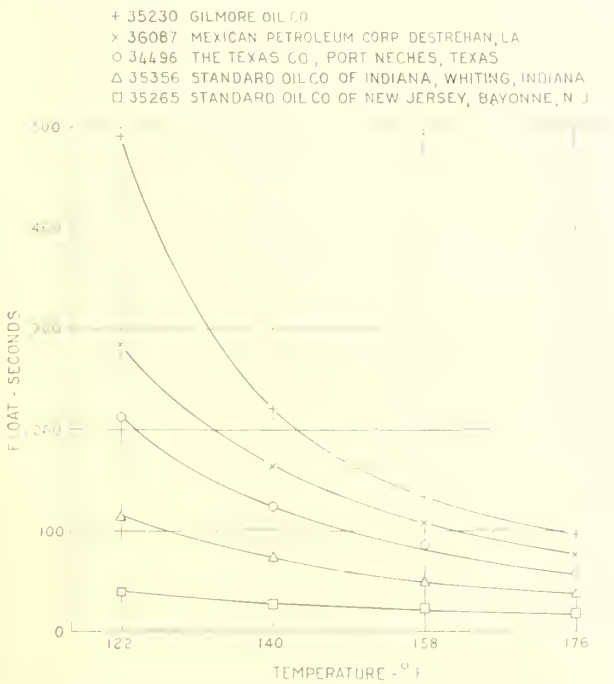

+ 35230 GILMORE OIL CO
× 36087 MEXICAN PETROLEUM CORP. DESTREHAN, LA
○ 34496 THE TEXAS CO., PORT NECHES, TEXAS
△ 35356 STANDARD OIL CO OF INDIANA, WHITING, INDIANA
□ 35265 STANDARD OIL CO OF NEW JERSEY, BAYONNE, N J
FLOAT - SECONDS
100
0
122
140
158
176
TEMPERATURE - °F

FIGURE 9.—RELATION BETWEEN TEMPERATURE AND FLOAT-TEST VALUES FOR MATERIALS FOR HOT SURFACE TREATMENT.

The one outstanding feature developed by these consisteney tests is that although the materials tested are often considered as fluid asphaltic materials, 14 of the group tested gave penetrations at 77° F. (100 grams, 5 seconds), from 264 to as low as 121, and that only 10 may be considered as liquid bituminous materials as such materials are defined by the American Society for Testing Materials. According to this definition only those materials having a penetration at 77° F. under a load of 50 grams applied for 1 second, of more than 350, are liquids. The viscosity determination at high temperatures for materials for hot-surface treatment is unnecessary. A float test could be used, but for soft semisolid asphalts a penetration test at 77° F. under a load of either 50 or 100 grams seems to offer the better means for laboratory control and avoids the use of viscosity tests at high temperatures.

CONCLUSIONS

1. There is need for tests at a number of temperatures to adequately cover the wide range in consisteney of liquid asphaltic road materials.

2. The use of 140° F. and 180° F. should prove satisfactory as control temperatures in Furol viscosity determinations for the more viscous liquid asphaltic road materials.

3. When practicable the temperature designated for control should give a Furol viscosity of less than 500 seconds. However, the more viscous materials of the rapid-curing type should not be tested for viscosity at a temperature higher than 140° F.

4. Many so-called liquid asphaltic road materials, which are used in hot surface-treatment work, and which are designated as hot oils, 90 and 95 percent road oils, etc., are in fact semisolid asphaltic materials. These grades of material probably can be controlled best for consistency by either a float test at a low temperature or by a penetration test at 77° F. under a load of 50 or 100 grams.

5. Tests at a temperature of approximately 210° F. can be omitted readily from specifications for liquid asphaltic road materials.

(Continued from p. 211)

stronger and better suited to severe traffic conditions such as were imposed.

17. Sand considerably coarser than is generally employed in asphalt pavement construction, was satisfactorily used in the stable, dense, coarse-graded asphaltic concretes of the first series of tests.

18. No influence of the consistency of the asphalt cement upon the service characteristics of the coarse-graded asphaltic concretes was apparent. Stable mixtures were laid with asphalts ranging from 45 to 75 penetration.

19. The five sections of asphaltic concrete of the second series of tests satisfactorily carried bitumen to the extent of 85 percent to 95 percent of the volume of the aggregate voids. Aggregate voids were determined for slabs removed from the traffic lanes after the service tests had been completed.

20. The stabilities determined by the roller stability machine conform to the observed service characteristics of the asphaltic concretes tested in the second series of tests. This apparatus constitutes a means by which mixtures of this type may be conveniently studied and compared.

LIMITED APPLICATION OF TEST RESULTS EMPHASIZED

Attention has been called to the reasons for these tests and to the conditions under which they were conducted, but it may be well to emphasize their limited scope and to warn against unconsidered application of the data and observations presented. At no time was it anticipated that a complete method of bituminous pavement design would be developed from these tests. It was desired merely to determine under controlled traffic conditions the relative traffic resisting capacities of a wide range of mixture compositions for correlation with laboratory stability tests which were in process of development. This aim was realized but the data collected were naturally studied with relation to existing theories.

There are several reasons why the indications of these experiments cannot be applied indiscriminately to all problems of mixture design. The property of stability is but one characteristic which a mixture should possess. To design for the highest possible stability may result in the adoption of a mixture which is unduly expensive, greatly deficient in bitumen or otherwise unbalanced, and consequently not the best suited to the specific conditions under which it is to be used. Very high stabilities, such as those apparently necessary to resist the exceptionally severe traffic of these tests, are not generally essential. It was found that a minimum stability of about 2,000 pounds was required on certain heavy-traffic New York City streets and under light to moderate traffic on country roads 1,000 pounds stability, or even a little less, has proved adequate to resist deformation. In such cases as the latter an attempt to attain unnecessarily high stabilities might yield pavements which would crack excessively or which would be undesirably high in voids.

Materials from different sources vary widely in their capacity to impart stability to mixtures. Since these tests were conducted, research has demonstrated that the sands used in these tests possess greater stabilities than those from many other sections of the country. This characteristic is not a function of gradation, so that sands selected at random but duplicating the mesh composition of these experimental mixtures cannot be similarly proportioned in surface mixtures and trusted to develop corresponding stabilities.

Therefore the value of this study does not rest in the actual pounds of stability, percentage of voids, or other characteristics of the mixtures described, but rather in the indications of interrelationship which exist and which must be considered, weighed, and evaluated with reference to the controlling factors of traffic, materials, climate, etc., involved in any individual design problem.

A LABORATORY TRAFFIC TEST FOR LOW-COST ROAD TYPES

THE Bureau of Public Roads has recently built and placed in operation at the Arlington Experiment Farm a small circular test track for applying, in the laboratory, traffic tests to sections of highway surfaces. The test was designed primarily for the study of low-cost bituminous types, but it is believed that it may be adapted for other studies such as, for instance, subgrade stabilization, motor-vehicle tire wear, etc.

The track consists of an annular concrete trough 12 inches deep, 18 inches wide, and 12 feet in diameter at the center line. The depth is sufficient to permit the use of various combinations of base materials beneath the bituminous test surfaces. Along the smaller circumference of the trough in which the test sections are held, and cast integrally with it, is another trough 3 inches wide and 13 inches deep intended to be used as a reservoir for the introduction of water into the base material under the test surfaces through small openings at the base of the partition wall. By this arrangement the track may be flooded or the water may be introduced through capillarity.

Two full-size automobile wheels provide the traffic for the tests. These wheels are fixed to the two ends of a rigid structural member which is rotated in a horizontal plane by a vertical shaft in the pedestal at the center of the track. The upper end of this shaft is squared and on it rides a freely sliding square nut mounted in trunnions in the cross member. This arrangement causes a constant wheel load (that due to the weight of the wheels, tires, and cross member) to be applied at all times regardless of the irregularities of the test surfaces. At present this load amounts to about 800 pounds per tire. Although the distance between the two test wheels is fixed, a handwheel adjustment is provided which shifts the position of the square nut with respect to the midpoint of the cross member and enables the operator to place the path of either wheel at any point on the test surface.

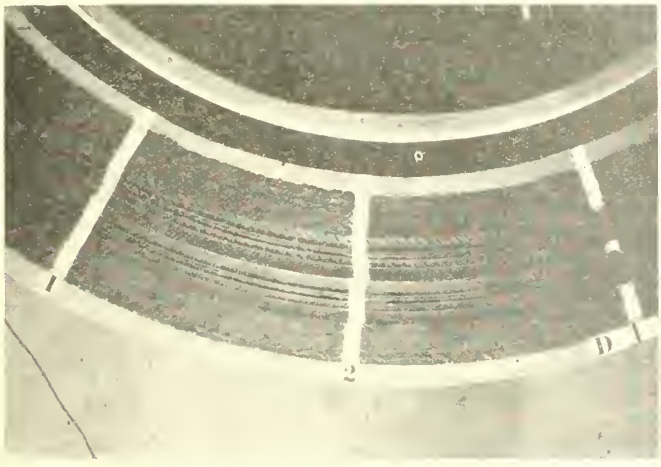
An electric motor operating through a 3-step cone pulley and a worm reduction drives the vertical shaft at the center of the track.

The test wheels may be operated at speeds of $4\frac{1}{2}$, 6, or 9 miles per hour as desired. The low speed has proved to be most convenient when distributed traffic for compacting the surface is required. For the testing of the completed surfaces, concentrated traffic and the highest speed are used.

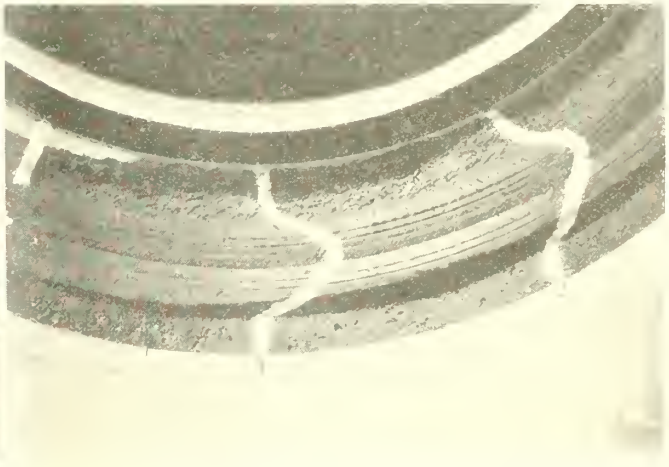
The number of trips made by each wheel is recorded by an electrical contact mechanism on the central vertical shaft operating a magnetic revolution counter at a point outside the track. In addition to this record, the data being collected include the corresponding behavior of the material under test, the density of the surface before and after test, oil migration, water content, and amount of material lost due to raveling. It is hoped that this information will make possible the evaluation of the important factors affecting the behavior of oiled aggregate mixtures.

The apparatus is now being used to investigate the effect of the percentage and consistency of the bituminous material on the durability and stability of mixtures with one type and grading of aggregate. A later phase of this first series of tests will involve a study of the effect of capillary water on the same mixtures. Various other factors influencing the behavior of different types of bituminous surface will be studied.

From the preliminary work which has been done up to this time it appears that the apparatus will provide a very useful method of studying some of the many factors involved in the performance of low-cost bituminous surfaces.



The asphaltic material in this section has a Saybolt-Furol viscosity at 122° F. of 1,230.



The asphaltic material in this section has a Saybolt-Furol viscosity at 122° F. of 89.

EFFECT OF OIL CONSISTENCY ON THE STABILITY OF THE OILED AGGREGATE SURFACE UNDER TEST TRAFFIC.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS I—PROJECTS ON THE FEDERAL-AID HIGHWAY SYSTEM
OUTSIDE OF MUNICIPALITIES
AS OF DECEMBER 31,1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
Alabama	$ 4,185,067	$	$			$ 3,978,158.68	$ 2,190,403.56	$ 1,787,755.12	18.7	211.2	$ 525,638.08	57.8	$ 1,469,025.36
Arizona	3,804,731	50,754.77	50,754.77		.2	3,051,450.72	2,738,158.16		20.5	210.1	474,965.53	65.1	540,852.54
Arkansas	3,374,167					1,095,153.69	739,891.53	355,262.16	27.6	34.1	582,978.19	43.3	2,051,297.28
California	7,803,677					6,621,812.82	5,424,473.42		17.5	215.0	1,189,616.95	39.0	1,189,586.63
Colorado	3,437,265	198,864.32	198,864.32		14.8	1,652,364.39	1,652,364.39		36.9	83.4	223,152.15	4.8	1,362,884.14
Connecticut	1,404,213					186,337.04	186,337.04		19.3	3.0	1,217,874.96	27.7	1.00
Delaware	909,544	133,532.30	133,532.30		15.4	362,183.80	362,183.80		32.6	7.3	393,535.00	11.4	20,292.90
Florida	2,615,917					2,239,498.29	1,468,319.66	770,568.53	25.1	83.9	569,730.95	20.3	577,866.39
Georgia	5,045,592					781,431.96	781,431.96		32.1	30.8	1,728,129.94	109.0	2,536,030.10
Idaho	2,243,125	51,214.29	50,084.37		18.9	1,582,189.31	1,568,586.16		49.1	111.7	162,825.41	13.4	461,629.06
Illinois	4,431,348					630,863.60	630,863.60		4.1	17.0	571,573.50	9.9	3,228,910.90
Indiana	4,717,786					2,008,101.96	2,008,101.96		26.9	67.5	1,124,202.64	32.2	1,585,481.40
Iowa	5,027,830	729,099.63	704,400.00		30.1	3,944,046.72	3,716,000.00		29.2	209.4	465,830.00	22.6	141,600.00
Kansas	5,044,802	21,817.08	21,817.08		15.3	2,543,082.43	2,489,323.70		30.5	269.2	2,533,661.22	129.6	
Kentucky	3,608,332	51,941.61	51,941.61		1.6	1,554,677.14	1,554,677.14		29.8	123.3	1,070,787.76	78.8	930,925.49
Louisiana	2,914,295	12,626.07	12,626.07		.4	549,193.11	549,193.11		49.2	23.8	1,614,570.13	37.8	737,905.69
Maine	1,684,959					1,177,533.70	1,141,966.75		34.8	35.5	201,928.52	7.1	341,063.73
Maryland	1,782,263					156,305.93	146,657.18		21.1	4.8	644,837.85	10.0	990,767.97
Massachusetts	1,932,950	153,936.88	83,952.64	$ 69,984.24	4.2	1,155,771.39	860,635.58	295,135.81	14.5	32.8	66,479.60	.7	921,882.18
Michigan	5,094,491	35,500.00	35,500.00		1.2	1,636,150.00	1,636,150.00		37.3	106.1	2,618,841.00	96.0	804,000.00
Minnesota	5,115,153	996,229.46	996,229.46		202.0	1,740,834.16	1,740,834.14		41.1	258.9	817,287.40	205.6	1,560,802.00
Mississippi	3,489,337					796,451.01	401,955.65	355,863.42	26.4	42.1	1,614,345.35	151.9	1,473,036.00
Missouri	6,090,153	92,412.78	84,236.45		5.7	2,863,692.42	2,575,645.77		52.5	93.2	2,095,993.03	73.3	1,354,277.75
Montana	4,463,849	176,156.03	176,156.03		24.0	3,334,381.70	3,154,381.70	100,000.00	29.3	247.0	575,446.01	46.8	557,865.26
Nebraska	3,914,481	363,958.41	258,455.96		138.7	3,638,894.08	2,798,913.98	108,538.00	37.7	167.1	743,211.79	63.6	113,899.27
Nevada	2,909,387	84,617.72	84,617.72		32.2	1,552,731.71	1,552,731.71		21.6	154.6			1,272,037.57
New Hampshire	954,919					515,633.37	515,633.37		27.2	9.1	109,759.09	1.5	329,526.54
New Jersey	3,065,137					791,445.80	791,445.80		10.4	9.7	433,517.15	10.8	1,840,174.05
New Mexico	2,896,467	419,595.83	419,595.83		68.9	1,964,317.48	1,964,317.48		25.9	164.5			512,553.69
New York	10,830,099	117,100.00	117,100.00		3.8	8,552,060.00	7,989,020.00	315,000.00	20.0	186.9	2,228,660.00	44.1	495,319.00
North Carolina	4,761,147	320,365.45	160,182.74	160,182.71	25.9	1,263,292.01	867,475.34	395,816.67	23.5	135.2	1,227,887.28	207.2	2,505,601.64
North Dakota	2,902,224	503,828.71	503,828.71		249.7	773,152.37	773,152.37		8.4	212.7	1,237,048.11	355.1	388,194.81
Ohio	6,968,066	99,220.00	99,220.00		1.3	5,576,960.00	5,576,960.00		31.1	147.8	908,510.00	23.9	383,376.00
Oklahoma	4,608,399					3,151,965.53	3,151,965.53		28.7	183.9	800,793.16	76.3	655,840.31
Oregon	3,053,448	198,986.16	171,092.66		36.2	2,167,525.70	2,147,119.70		24.6	97.6	463,754.46	36.9	271,481.18
Pennsylvania	5,757,978					3,220,112.69	3,220,112.69		8.8	81.1	1,673,600.51	37.1	864,264.80
Rhode Island	999,354	18,830.63	18,830.63		.5	969,153.21	969,153.21		23.7	20.0			11,370.16
South Carolina	2,729,583	1,740.69	1,740.69			1,281,635.82	1,278,906.20	2,729.62	26.4	142.0	728,171.83	58.3	720,764.28
South Dakota	3,005,739	802,741.64	802,741.64		105.0	1,121,651.99	1,083,711.21	37,940.78	6.2	206.6	416,778.24	45.7	702,507.91
Tennessee	4,246,309	131,887.64	72,591.00	59,296.64	5.1	1,930,521.70	1,413,800.96	516,720.74	17.5	63.6	1,419,893.89	69.2	1,340,023.15
Texas	12,122,012	152,449.73	126,731.89		39.9	6,288,047.46	5,129,350.89		22.0	570.7	2,405,079.12	274.4	4,460,850.10
Utah	2,097,354	773,633.06	773,633.06		97.5	910,466.51	910,466.51		38.7	69.2	413,254.43	23.7	
Vermont	931,919	47,983.81	47,983.81		6.1	639,978.50	639,978.50		32.0	30.5	67,220.10	4.3	176,736.59
Virginia	3,708,379	94,272.69	83,502.98	10,769.71	8.2	1,345,621.49	1,339,208.33	6,413.16	33.4	51.9	781,353.01	24.1	1,504,314.68
Washington	3,057,934	187,015.09	187,015.09		13.2	1,532,794.71	1,529,437.89	3,356.82	23.2	53.0	878,645.11	31.2	462,835.91
West Virginia	2,013,405	116,220.82	116,220.82		4.2	1,511,877.13	1,511,877.13		25.7	64.9	164,976.32	1.9	220,330.73
Wisconsin	4,862,441	298,477.25	288,174.42	1,190.00	15.7	2,438,924.84	2,417,424.84	21,500.00	37.6	100.3	530,917.03	38.3	1,625,924.71
Wyoming	2,250,663	543,414.19	457,300.00	60,000.00	107.4	1,491,217.76	1,417,600.00		16.2	302.0	118,182.61	29.2	257,580.39
District of Columbia													
Hawaii	1,683,956	25,988.65	12,994.33	12,994.32	.9	842,351.10	634,770.27	157,581.08	3.2	17.8	684,305.08	13.4	351,886.32
TOTALS	186,551,646	8,006,413.39	7,383,649.08	374,417.62	1,294.2	101,113,998.93	91,343,069.87	5,230,181.91	26.3	5,763.8	41,519,749.49	2,764.3	46,305,177.56

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS II—NATIONAL RECOVERY MUNICIPAL HIGHWAY PROJECTS
(ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES)

AS OF DECEMBER 31, 1933

STATE	NATIONAL RECOVERY FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	National recovery funds	Regular Federal aid	Mileage	Estimated total cost	National recovery funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage	
Alabama	$ 2,092,533					$ 173,815.13	$ 137,692.99	$ 34,122.14	55.2	5.3	$ 155,432.61	3.5	$ 1,799,437.40
Arizona	781,794					11,448.25	11,448.25		78.6	1.0	38,841.83	1.2	731,503.92
Arkansas	1,687,084					201,302.27	153,847.30	47,454.97	13.6	5.5	325,045.22	9.0	1,208,191.48
California	3,901,839					1,014,064.74	873,360.38		12.9	17.7	1,302,468.65	15.2	1,726,009.97
Colorado	1,718,633	$ 128,443.81	$ 128,443.81		3.3	120,803.48	120,803.48		16.6	2.4	385,474.01	7.7	1,083,911.70
Connecticut	802,407					391,427.79	391,427.79		42.4	5.2	264,579.80	3.3	146,399.41
Delaware	454,772	77,384.90	77,384.90		1.5	149,600.00	149,600.00			1.1			227,787.10
Florida	1,307,959					476,134.13	298,238.21	177,895.92	15.1	5.9	482,541.59	3.8	527,179.20
Georgia	2,724,620					62,205.00	62,205.00		17.7	3.1	563,180.25	20.5	2,099,234.75
Idaho	1,121,562	1,750.35	1,750.35		.6	313,857.19	308,462.79		14.9	6.5	61,135.69	1.8	750,213.17
Illinois	6,877,199	45,749.72	45,749.72		.6	1,643,952.03	1,643,952.03		4.9	17.9	3,873,452.74	35.9	1,314,044.51
Indiana	4,818,165					24,698.54	24,698.54			.4	103,720.85	2.1	4,689,745.61
Iowa	2,315,585	181,286.46	169,850.00		5.5	708,430.92	665,450.00		25.8	17.0	621,200.00	16.1	1,354,085.00
Kansas	2,522,401	2,114.16	2,114.16		.8	359,636.94	359,636.94		7.2	5.3	2,160,649.90	10.6	
Kentucky	2,029,687					20,886.00	20,886.00		76.6	.6	166,239.55	3.7	1,842,561.45
Louisiana	1,457,148					329,960.57	329,960.57		39.7	6.5	116,195.40	5.7	1,010,992.03
Maine	842,479	78,746.59	78,746.59		1.6	340,046.25	340,046.25		30.3	8.5	88,884.90	1.6	334,801.26
Maryland	891,132					16,788.76	12,638.76		79.1	.6			878,493.24
Massachusetts	4,136,382	53,896.49	30,896.49	$ 23,000.00	1.4	1,779,131.94	1,765,031.94	14,100.00	15.0	11.2	355,184.22	1.9	1,985,269.35
Michigan	4,457,679	180,700.00	180,700.00		2.3	460,770.00	458,620.00		5.2	10.5	1,278,651.00	11.8	2,539,708.00
Minnesota	3,410,102	425,560.77	425,560.77		27.7	683,767.45	683,767.45		54.7	30.3	571,860.46	23.2	1,728,913.32
Mississippi	1,744,669					143,412.49	86,345.12	57,067.37	42.9	3.5	310,934.06	16.0	1,347,389.82
Missouri	3,045,977	2,596.65	1,948.52		.2	634,565.30	622,092.91		36.0	7.0	694,172.95	16.1	1,726,862.62
Montana	1,115,962	2,878.39	2,878.39		.1	91,327.03	91,327.03		46.0	3.7	325,833.34	11.6	695,873.24
Nebraska	1,957,240	29,267.12	28,577.53		3.6	906,132.56	906,132.56		37.3	15.6	137,331.55	3.6	915,198.06
Nevada	500,051					50,304.27	50,304.27		89.5	1.2			449,746.73
New Hampshire	477,460					301,104.78	301,104.78		11.3	6.4	79,020.57	1.6	97,334.55
New Jersey	3,217,442					1,718,880.86	1,718,880.86		21.8	13.3	125,110.11	1.1	1,373,451.03
New Mexico	1,448,234					173,201.60	173,201.60		19.6	4.8	413,671.68	6.7	861,360.72
New York	7,837,365	115,700.00	115,700.00		2.2	4,234,800.00	4,153,900.00		15.0	32.6	2,713,185.00	22.6	855,080.00
North Carolina	2,380,573	42,316.20	42,316.20		3.3	67,744.99	67,744.99		51.7	5.4	120,728.85	9.9	2,143,782.96
North Dakota	1,451,112	1,899.97	1,899.97		1.0	47,202.69	47,202.69		36.0	6.1	179,202.44	7.2	1,222,806.40
Ohio	4,645,378	47,460.00	39,985.00		1.6	1,109,420.00	1,026,920.00		23.3	12.9	507,965.30	8.1	3,071,408.00
Oklahoma	2,304,200					184,648.94	184,648.94		20.6	3.8	331,288.79	11.1	1,738,262.27
Oregon	1,526,724					494,913.03	494,913.03		10.1	7.8	286,840.73	8.6	744,961.34
Pennsylvania	5,416,051	46,325.04	46,325.04		1.7	683,149.42	680,938.71		18.7	15.6	1,900,336.64	10.0	2,788,420.61
Rhode Island	499,677					60,871.25	60,871.25		9.9	1.2	234,011.36	3.0	204,794.39
South Carolina	1,364,791	24,919.48	24,919.48		1.7	67,316.48	66,911.47	405.01	13.5	6.8	253,213.51	10.3	1,014,746.44
South Dakota	1,502,870	69,938.16	69,938.16		4.2	159,132.03	159,132.03		9.4	6.9	128,621.53	4.8	1,145,176.28
Tennessee	2,123,155	19,992.06	19,992.06		1.1	111,814.03	111,814.03		5.4	3.0	330,765.41	4.7	1,611,083.50
Texas	6,061,006	768.11	768.11		1.3	770,728.17	672,514.39		18.4	34.5	1,139,111.70	44.3	4,258,611.80
Utah	1,048,677	351,502.28	351,425.28		8.8	167,620.89	166,661.89		31.2	4.3	80,689.92	2.1	443,899.91
Vermont	470,628	3,854.07	3,854.07		.6	258,945.10	256,943.99		20.2	7.4	169,994.42	4.0	39,835.52
Virginia	1,854,189	20,431.84	11,671.35	8,760.49	1.4	310,853.87	310,853.87		29.9	6.3	682,529.12	8.7	849,134.66
Washington	1,877,571	65,733.19	65,733.19		2.3	312,368.37	312,368.37		17.3	6.6	1,005,446.77	15.4	493,022.67
West Virginia	1,342,270					103,573.64	103,573.64		25.1	1.9	407,243.17	7.8	831,453.19
Wisconsin	2,431,220	131,355.96	131,355.96		4.4	646,155.41	646,155.41		32.0	16.6	260,858.86	6.1	1,392,849.77
Wyoming	1,125,332	53,504.59	53,004.59		1.1	123,405.50	123,405.50			3.3	277,444.08	7.1	671,477.73
District of Columbia	959,235	508,017.51	508,017.51		3.2	278,337.00	278,337.00			.2	172,880.49	1.3	
Hawaii													
TOTALS	112,579,821	2,714,093.97	2,661,507.60	31,760.49	89.1	23,193,757.08	22,686,975.00	331,045.41	20.3	402.0	26,258,772.42	472.4	60,972,565.98

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS III—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF DECEMBER 31, 1933

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	Public works funds	Mileage	Estimated total cost	Public works funds allotted	Percentage completed	Mileage	Public works funds allotted	Mileage		
Alabama	$ 2,092,533				$ 40,700.00	$ 40,700.00	63.9	6.0	$ 139,527.30	12.7	$ 1,953,005.70	Alabama
Arizona	625,435				9,586.90	9,586.90		.2	329,288.60	26.1	255,446.40	Arizona
Arkansas	1,687,084								292,675.46	14.8	1,384,821.64	Arkansas
California	3,901,838				812,952.00	662,730.28	11.5	75.5	1,252,573.43	48.4	1,986,534.29	California
Colorado	1,718,632				689,262.42	689,262.42	41.9	139.9	234,602.89	42.5	794,766.69	Colorado
Connecticut	659,120				160,281.50	160,281.50	3.7	3.1	496,532.44	11.4	2,306.06	Connecticut
Delaware	454,772								138,752.00	1.8	316,020.00	Delaware
Florida	1,307,958				887,239.72	837,239.72	8.0	61.9	305,473.46	6.6	115,244.82	Florida
Georgia	2,320,973								319,912.59	26.2	2,001,060.41	Georgia
Idaho	1,121,562	$ 11,625.34	$ 7,523.08	2.8	813,681.26	783,778.86	33.8	121.7			330,260.06	Idaho
Illinois	6,262,223				464,038.89	464,038.89	11.4	69.9	2,120,622.90	141.4	3,677,561.21	Illinois
Indiana	501,892				165,000.00	165,000.00	2.4	48.1	151,464.91	11.6	185,427.09	Indiana
Iowa	2,212,245	46,834.43	44,100.00	7.6	450,456.45	429,700.00	42.6	36.9	322,100.00	42.8	1,416,345.00	Iowa
Kansas	2,522,401				395,339.02	395,339.02	2.8	23.8	2,127,061.98	148.7		Kansas
Kentucky	1,879,340				626,409.65	626,409.65	5.3	72.4	1,197,426.54	147.8	55,503.81	Kentucky
Louisiana	1,457,148				61,627.65	61,627.65	86.0	2.3	488,281.84	27.2	907,238.51	Louisiana
Maine	842,479	575,255.93	571,104.50	69.4	304,268.27	259,703.26	73.9	28.4	10,634.85	.2	1,036.39	Maine
Maryland	891,132				18,674.35	18,674.35		1.5	286,111.27	21.0	586,346.38	Maryland
Massachusetts	527,768				367,127.97	367,127.97	44.1	11.5	104,669.95	3.7	55,970.08	Massachusetts
Michigan	3,184,057				1,116,550.00	1,116,550.00	2.5	88.3	1,577,800.00	136.0	489,707.00	Michigan
Minnesota	2,131,314	28,119.39	28,119.39	21.2	1,195,395.21	1,107,217.08	21.0	168.6	399,097.39	76.1	596,880.14	Minnesota
Mississippi	1,744,669				435,000.00	435,000.00	22.1	54.6			1,309,669.00	Mississippi
Missouri	3,045,076				1,258,322.78	1,258,322.78	13.5	245.6	1,323,794.68	184.1	462,958.54	Missouri
Montana	1,859,937				231,205.86	231,205.86	7.4	23.9	691,116.50	83.2	937,614.64	Montana
Nebraska	1,957,240				1,605,898.84	1,604,036.52	17.2	249.7	36,078.47	4.5	317,125.01	Nebraska
Nevada	1,136,479				348,863.14	348,863.14	37.8	52.4	493,777.31	44.6	293,838.55	Nevada
New Hampshire	477,460				450,313.37	449,645.19	25.6	21.0			27,814.81	New Hampshire
New Jersey	63,460				56,550.52	56,550.52		.5			6,909.48	New Jersey
New Mexico	1,418,234				1,239,750.00	1,239,750.00	8.5	747.1	195,750.00	184.5	12,734.00	New Mexico
New York	3,662,137	88,300.00	88,300.00	1.9	3,333,863.19	2,994,388.19	14.9	80.2	534,575.00	17.5	44,873.81	New York
North Carolina	2,380,573	61,925.32	61,925.32	7.1	879,402.13	879,402.13	10.9	80.1	217,406.02	16.0	1,221,839.53	North Carolina
North Dakota	1,451,112								46,035.01	10.4	1,405,076.99	North Dakota
Ohio	3,871,148	40,500.00	40,500.00	16.2	2,118,510.00	2,013,074.90	15.3	223.4	1,283,690.00	57.9	533,883.10	Ohio
Oklahoma	2,304,199								98,609.03	.9	2,205,589.97	Oklahoma
Oregon	1,526,724				219,638.32	218,638.32	4.6	9.9	986,385.11	73.9	321,700.57	Oregon
Pennsylvania	7,716,975	6,570.00	6,570.00	.7	5,126,653.92	5,126,653.92	20.4	520.9	1,891,484.43	158.3	692,266.65	Pennsylvania
Rhode Island	499,677				7,631.36	7,631.36		1.1	172,014.08	15.2	320,031.56	Rhode Island
South Carolina	1,364,791				1,004,981.24	1,004,981.24	7.9	121.6	278,547.60	23.4	81,262.16	South Carolina
South Dakota	1,502,870				82,493.81	82,493.81	77.6	30.9	91,472.22	34.9	1,328,903.97	South Dakota
Tennessee	2,123,155				293,139.51	293,139.51	6.8	32.6	601,278.55	54.3	1,228,736.94	Tennessee
Texas	6,061,006				2,826,638.25	1,995,337.93	23.0	331.4	1,874,059.44	326.8	2,191,608.63	Texas
Utah	1,048,677	61,543.36	61,543.36	8.0	731,982.71	731,982.71	23.9	151.4	18,942.55	.2	236,208.38	Utah
Vermont	465,026				248,334.26	233,799.39	19.2	22.6	109,057.75	9.5	122,168.86	Vermont
Virginia	1,854,189	5,000.00	5,000.00	3.7	1,058,285.93	1,058,285.93	25.3	158.7	295,423.21	31.0	495,479.86	Virginia
Washington	1,180,362	36,856.60	36,856.00	11.6	622,532.39	612,532.39	12.9	33.6	351,075.32	16.5	179,898.29	Washington
West Virginia	1,118,559				142,992.63	142,992.63	13.3	5.4	308,837.33	20.3	666,729.04	West Virginia
Wisconsin	2,431,220	26,056.01	26,056.01	.6	1,192,443.23	1,153,427.95	22.1	88.6	604,150.17	38.8	647,585.87	Wisconsin
Wyoming	1,125,332				555,157.53	548,801.49	22.8	89.2	106,499.61	7.7	470,030.90	Wyoming
District of Columbia	187,106	62,897.00	62,897.00	1.5	268,004.12	268,004.12	52.6	1.9	575,491.35	4.3	52,841.53	District of Columbia
Hawaii	959,234								180,042.06	4.9	7,063.94	Hawaii
TOTALS	94,868,533	1,051,483.38	1,040,494.66	152.3	34,967,180.30	33,233,909.48	18.7	4,338.3	25,660,202.60	2,370.6	34,933,926.26	TOTALS

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D.C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 10 cents.

DEPARTMENT BULLETINS

No. 136D . . Highway Bonds. 20 cents.

No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.

No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.

No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 660D . . Highway Cost Keeping. 10 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys. 20 cents.

No. 265T . . Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS CIRCULARS

No. 62MC . . Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects. 5 cents.

No. 93MC . . Direct Production Costs of Broken Stone. 25 cents.

MISCELLANEOUS PUBLICATION

No. 76MP . . The results of Physical Tests of Road-Building Rock. 25 cents.

No. ——— . . Federal Legislation and Regulations Relating to Highway Construction. 10 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U.S. Bureau of Public Roads, Willard Building, Washington, D.C.

CURRENT STATUS OF NATIONAL RECOVERY ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

SUMMARY OF CLASSES I, II, AND III

AS OF DECEMBER 31,1933

STATE	TOTAL APPORTIONMENT OF NATIONAL RECOVERY FUNDS	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF NATIONAL RECOVERY FUNDS AVAILABLE FOR ALL NEW PROJECTS
		Total cost	National recovery funds	Regular Federal aid	Mileage	Estimated total cost	National recovery funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	National recovery funds allotted	Mileage	
Alabama	$ 8.370.133	$	$			$ 4,151.973.81	$ 2.328.096.55	$ 1.821.877.26	20.8	216.5	$ 820.567.99	74.0	$ 5.221,468.46
Arizona	5.211.960	50.754.77	50,754.77		.2	3,103.598.97	2.790.306.41		21.4	217.1	843.095.96	92.4	1.527.802.86
Arkansas	6.748.335					1.306.042.86	903.325.73	402.717.13	24.9	39.8	1.200.698.87	67.1	4.644,310.40
California	15.607.354					8,448.829.56	6.960.564.08		16.3	308.2	3.744.659.03	102.6	4.902,130.89
Colorado	6.874.530	327.308.13	327.308.13		18.1	2.462.430.29	2.462.430.29		37.3	225.7	843.229.05	55.0	3.241,562.53
Connecticut	2.865.740					738.046.33	738.046.33		28.2	11.3	1.973.987.20	42.4	148,706.47
Delaware	1.819.088	210.917.20	210.917.20		16.9	511.783.80	511.783.80		23.1	8.4	532.287.00	13.2	564,100.00
Florida	5.231.834					3.602.872.14	2.653.797.59	948,464.45	18.2	151.7	1.357.746.00	30.7	1.220.290.41
Georgia	10.091.185					843.636.96	843.636.96		31.1	33.9	2.611.222.78	155.7	6.636.325.26
Idaho	4.486.249	64.589.98	59.357.80		22.3	2.709.727.76	2r660.327.81		40.6	239.9	223.961.10	15.2	1.542.102.29
Illinois	17.570.770	45.749.72	45.749.72		.6	2.733.354.52	2.738.354.52		5.8	104.8	6.565.649.14	187.2	8.220.516.62
Indiana	10.037.843					2.197.300.50	2.197.300.50		24.3	116.0	1.379.388.40	45.9	6,460.654.10
Iowa	10.055.660	957.220.52	913,350.00		43.2	5,102,934.09	4.811.150.00		30.0	263.3	1.414.130.00	81.5	2,912.030.00
Kansas	10.089.604	23.931.24	23.931.24		16.1	3,298.058.39	3.241.299.66		24.5	298.3	6.821.373.10	308.9	
Kentucky	7.517.359	51.941.61	51.941.61		1.6	2.201.972.79	2.201.972.79		23.3	196.3	2.434.453.85	230.3	2.828.990.75
Louisiana	5.828.591	12.626.07	12.626.07		.4	940.781.33	940.781.33		48.3	32.6	2.219.047.37	70.7	2.656.136.23
Maine	3.369.917	654.002.52	649.351.09		71.0	1.821.848.22	1.741.716.26		39.7	72.4	301.448.27	8.9	676.901.38
Maryland	3.564.527					191.769.04	177.970.29		23.0	6.9	930.949.12	31.0	2.455.607.59
Massachusetts	6.597.100	207.833.37	114.849.13	$ 92.984.24	5.6	3,302.031.30	2.992.795.49	309.235.81	18.4	55.5	526.333.77	6.3	2.963.121.61
Michigan	12.736.227	216.200.00	216.200.00		3.5	3,213.470.00	3.211.320.00		20.6	205.2	5.475.292.00	243.8	3.833.415.00
Minnesota	10.656.569	1.449.909.62	1.449.909.62		250.9	3,619.996.82	3.531.318.67		37.5	457.8	1.788.245.25	304.9	3.886.595.46
Mississippi	6.978.675					1.374.863.50	923.300.77	412.930.79	25.9	100.2	1.925.279.41	167.9	4.130.094.32
Missouri	12.180.306	95.009.43	66.184.97		5.9	4.756.580.50	4.456.061.46		39.2	345.8	4.113.960.66	273.5	3.544.098.91
Montana	7.439.748	179.034.42	179.034.42		24.1	3.656.914.59	3.476.914.59	100.000.00	28.3	274.6	1.592.445.85	141.6	2.191.353.14
Nebraska	7.828.961	393.225.53	287.033.79		142.3	6,150.925.48	5.309.083.06	108.538.00	31.4	432.4	886.621.81	71.7	1.346.222.34
Nevada	4.545.917	84.617.72	84.617.72		32.2	1.951.899.12	1.951.899.12		26.2	208.2	493.777.31	44.6	2.015.622.85
New Hampshire	1.909.839					1.267.051.52	1.266.383.34		22.8	36.5	188.779.76	3.1	454.675.90
New Jersey	6.346.039					2,566.877.18	2.566.877.18		17.8	23.5	558.627.26	11.9	3.220.534.56
New Mexico	5.792.935	419.595.83	419.595.83		68.9	3.377.269.08	3.377.269.08		19.2	916.4	609.421.68	191.2	1.386.648.41
New York	22.330.101	321.100.00	321.100.00		7.9	16,170.723.19	15,137.308.19	315.000.00	17.6	299.7	5.476.420.00	84.2	1.395.272.81
North Carolina	9.522.293	424.606.97	264.424.26	160.182.71	36.3	2.210.439.13	1.814.622.46	395.816.67	18.5	220.7	1.566.022.15	233.1	5.877.224.13
North Dakota	5.304.448	505.728.68	505.728.68		250.7	820.355.06	820.355.06		10.0	218.8	1.462.286.06	372.7	3.016.078.20
Ohio	15.484.592	187.180.00	179.705.00		19.1	8.804.890.00	8.616.954.90		26.4	384.1	2.699.265.00	89.9	3.988.667.10
Oklahoma	9.216.798					3,336.614.47	3.336.614.47		28.3	187.7	1.280.690.98	88.3	4,599.492.55
Oregon	6.106.896	198.986.16	171.092.66		36.2	2.882.077.05	2.860.671.05		20.6	115.3	1.736.989.30	119.4	1.338.142.99
Pennsylvania	18.391.004	52.895.04	52.895.04		2.4	9.029.916.03	9.027.705.32		16.1	617.6	5.465.391.58	225.4	4.345.012.06
Rhode Island	1.998.703	18.830.63	18.830.63		.5	1.037.655.82	1.037.655.82		22.7	22.3	406.025.44	18.2	536.196.11
South Carolina	5.459.165	26.660.17	26.660.17		1.7	2.353.933.54	2.350.798.91	3.134.63	18.1	270.4	1.264.933.04	92.0	1.816.772.88
South Dakota	6.011.479	872.679.80	872.679.80		109.2	1.363.277.83	1.325.337.05	37.940.78	11.0	244.4	636.873.99	85.4	3.176.583.16
Tennessee	8.492.619	151.879.70	92.583.06	59.296.64	6.2	2.335.475.24	1.818.754.50	516.720.74	15.1	99.2	2.401.437.85	128.2	4.179.843.59
Texas	24.241.024	153.217.84	127.500.00		41.2	9.885.413.88	7.797.203.21		21.9	936.6	5.408.250.26	645.5	10.911.070.53
Utah	4.194.708	1.186.678.70	1.186.601.70		114.3	1.310.070.11	1.809.111.11		32.1	225.4	518.886.90	28.0	680.108.29
Vermont	1.867.573	51.837.88	51.837.88		6.7	1.146.357.86	1.130.721.88		26.7	60.5	346.272.27	17.5	338.740.97
Virginia	7.416.757	119.704.53	100.174.33	19.530.20	13.3	2.714.761.29	2.708.348.13	6.413.16	29.8	216.9	1.759.305.34	63.8	2.848.929.20
Washington	6.115.867	289.604.88	289.604.28		27.1	2.467.695.47	2.454.338.65	3.356.82	20.0	93.2	2.236.167.20	63.1	1.135.756.87
West Virginia	4.474.234	116.220.82	116.220.82		4.2	1.758.443.40	1.758.443.40		24.6	72.2	881.056.82	30.0	1.718,512.96
Wisconsin	9.724.381	455.889.22	445.586.39	1.190.00	20.7	4.277.523.48	4.217.008.20	21,500.00	32.5	205.5	1.395.926.06	83.2	3.666.360.35
Wyoming	4.501.327	596.913.88	510,304.69	60.000.00	108.5	2,169.780.79	2,089,806.99		17.0	394.5	502.126.30	44.0	1.399.089.02
District of Columbia	1.918.469	570,914.51	570.914.51		4.7	546.341.12	546.341.12		25.8	2.1	748.371.84	5.6	52.841.53
Hawaii	1.871.062	25.988.65	12.994.33	12.994.32	.9	842.351.10	634.770.27	157.581.08	3.2	17.8	864.347.14	18.3	358,950.26
TOTALS	394.000.000	11.771.990.74	11.085.651.34	406.178.11	1,535.6	159.574.936.31	147,263.954.35	5.561.227.32	23.7	10.504.1	93.438.724.51	5.607.3	142.211,669.80

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VOL. 14, NO. 12 FEBRUARY 1934

THE NEW JERSEY HIGH-LEVEL VIADUCT

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

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G. P. St. CLAIR, *Editor*

Volume 14, No. 12 February 1934

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions

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A TIME STUDY OF TRAFFIC FLOW ON THE NEW JERSEY HIGH-LEVEL VIADUCT

Reported by LAWRENCE S. TUTTLE, Assistant Highway Economist, Division of Highway Transport, United States Bureau of Public Roads

TRAFFIC FLOWS FREELY ON THE VIADUCT WITH A CONSIDERABLE SAVING IN TIME.

THE RECENTLY completed High-Level Viaduct between Newark and Jersey City in New Jersey has created wide interest because of the magnitude of the project and the engineering problems involved in its construction. Less publicity has been given the economic problems. Costing $5,200,000 per mile, it is one of the most intensive pieces of highway construction in the world. A careful preliminary investigation [1] indicated that of the various economies which would be effected by the construction of the express highway from Elizabeth to the Holland Tunnel, of which the viaduct is a part, the most important was the saving in travel time. This paper presents an estimate of the annual vehicle time saved by the viaduct under present traffic conditions, prepared from field studies of travel time on this express highway before and after the opening of the viaduct. The studies were made during 1932 and 1933 by the Division of Highway Transport, Bureau of Public Roads, with the cooperation of the New Jersey State Highway Commission. The field work was performed as an unemployment relief project in connection with the New Jersey State-wide traffic survey. No attempt has been made to value vehicle time saved in view of the general lack of agreement among engineers and economists on this subject, but the estimate has been so arranged that reasonable values for time saved for the several classes of vehicles may be easily applied to obtain the monetary value of the time saved.

OLD ROUTE BADLY CONGESTED

The viaduct completes the express highway from Elizabeth to the Holland Tunnel. This highway is one of the most heavily traveled in the country, as it is the principal approach from the south and west to New York City via the Holland Tunnel and the George Washington Bridge. It also serves a large volume of local traffic between Jersey City and Newark. Figure 1 shows the location of the viaduct and the previous routing of traffic. Before the viaduct was opened, traffic was routed east along Communipaw Avenue to West Side Park in Jersey City and then north to Tonnelle Circle, a distance of 4.2 miles.

Traffic congestion on this ground-level route often reached serious proportions. While the roadway provided at least two lanes in either direction at its narrowest point, interruptions to traffic and points of conflict were numerous. These consisted of frequent openings of the drawbridges across the Passaic and Hackensack Rivers, and traffic interference at numerous intersections, particularly at the east and west ends of the Communipaw Avenue section of the route, and also where Newark Avenue crosses near Tonnelle Circle.

Congestion at these three intersections was aggravated by heavy turning movements, requiring almost continuous police control. To a lesser degree, Tonnelle Circle was also a source of delay. In anticipation of construction of the viaduct the latter was not designed to accommodate a large volume of traffic, and before the completion of the viaduct an intricate system of police control was necessary to prevent complete tie-ups.

Traffic counts indicated that the total yearly traffic at the Hackensack River Bridge during 1932 was about 14,600,000 vehicles. Average week-day traffic throughout the year was approximately 37,000 vehicles, while on Sundays more than 50,000 vehicles per day were commonly recorded.

The heaviest traffic occurred on Sunday evenings in the summer with pleasure traffic returning to New

[1] A Proposed New Highway from the Westerly end of the Vehicular Tunnel in Jersey City via Kearny and Newark to Elizabeth, January 1925. This report consists of two sections: A Study of the Economic and Other Conditions Which Show the Need of This Proposed New Highway and Which Govern and Control its Design, by F. Lavis, and Studies of Traffic Data and Economic Values, by S. Johannesson.

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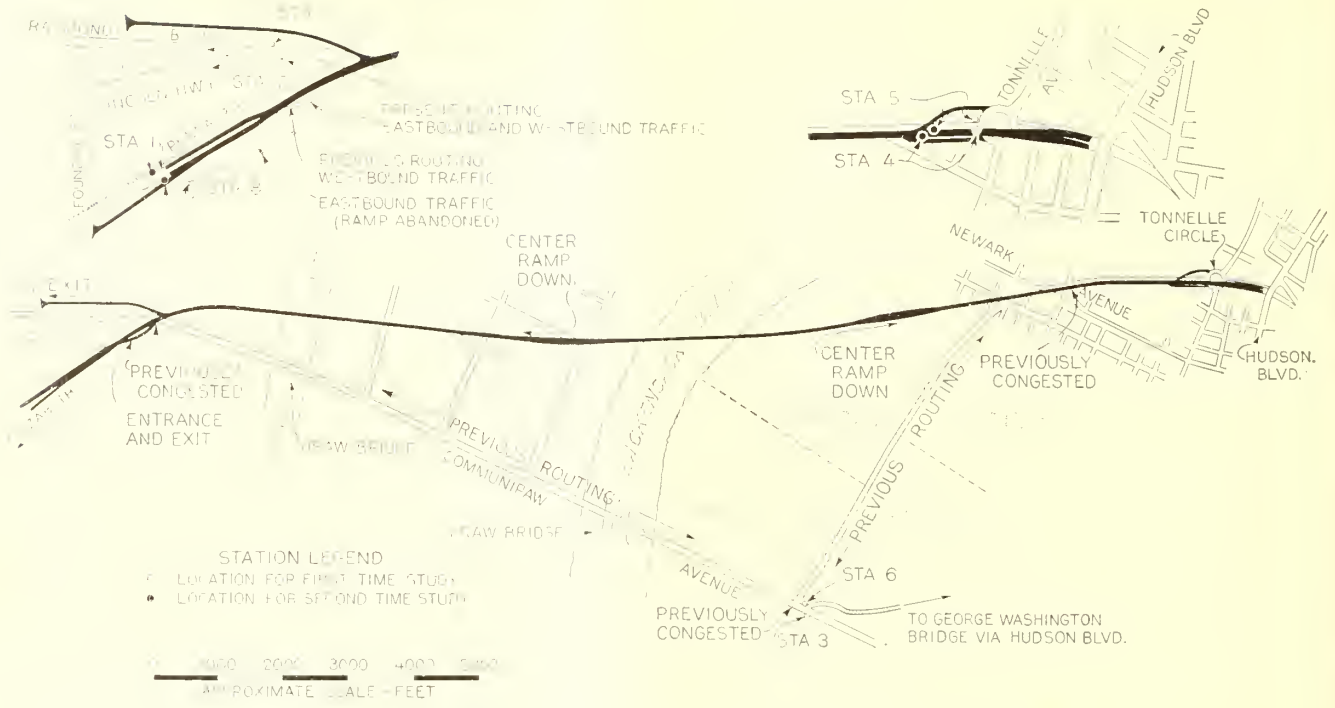


FIGURE 1.—MAP SHOWING OLD ROUTE (STIPPLED) AND NEW VIADUCT ROUTE.

York from the New Jersey shore resorts. Hourly traffic during this time frequently exceeded 2,000 vehicles in one direction. The viaduct which relieves this highly congested section is clear of all intersections at grade, provides at least two lanes for moving traffic in each direction, is high enough to eliminate the necessity of draw bridges and, in addition, has reduced the distance to 3.7 miles.

NEW FIELD SURVEY METHODS DEVELOPED

In order to estimate the vehicle time saved by the viaduct it was necessary to measure the average trip time on the old route and upon the viaduct between the same points. The difference in average trip time multiplied by the number of vehicles benefited represents the total vehicle time saved. The problem of measuring the average trip time on the old route was complicated by a wide variation in the frequency of bridge openings and in the volume of traffic throughout the day. It was necessary to time a sufficiently large number of trips to determine the effect of bridge openings and volume of traffic.

The method adopted involved the listing of vehicle license numbers at both ends of the route together with the time of passage of each vehicle to the nearest minute, and the subsequent matching of these license numbers so as to derive the trip time of each vehicle. The same method was followed to obtain the average trip time upon the viaduct. It was found that a record of only the last four digits of the license number, regardless of State of registration, was sufficient to identify a vehicle in all but a few exceptional cases.

Observation stations were located as shown in figure 1, each position having two stations, one for each direction of traffic. For the time study on the old route, station 8 and station 1 were located at the west end of the viaduct, east-bound vehicles being recorded at station 8 and west-bound vehicles at station 1. Stations 4 and 5, for east-bound and west-bound traffic, respectively, were placed at the opposite end of the route on the west side of Tonnelle Circle. Vehicles were timed from station 8 to station 4, and from station 5 to station 1, and the average trip time between these two pairs of stations was compared with subsequent time observations on the viaduct. A considerable volume of local traffic still uses Communipaw Avenue in traveling between Jersey City and Newark. In order to determine the time saved by this traffic as a result of diversion of through traffic to the viaduct, two additional pairs of stations, 3 and 6, and 7 and 2, were located at the east and west ends, respectively, of this section of the route. Vehicles were timed from station 7 to station 3, and from station 6 to station 2. For the time study of traffic on the viaduct only the two pairs of stations, 8 and 1 at the west end, and 4 and 5 at the east end, were used, with slight changes in location required by the new routing of traffic.

As might be supposed, the field personnel necessary to make such a study was large, particularly so on the old route, where eight stations were established. During periods of heavy traffic a maximum of 38 men were employed, including supervisors. The arrangement of personnel at the various stations was adapted to the physical conditions existing and to the volume of traffic to be recorded. At each station it was necessary to record the volume of traffic, to list separately the license numbers of passenger cars, trucks, and busses, and to call the time minute by minute. At the lighter traffic stations it was sometimes possible for one man to serve as timekeeper and truck and bus observer, but more frequently it was necessary to increase the personnel, especially the observers listing numbers. At the heavy traffic stations two passenger-car observers and two truck and bus observers were required. Usually these observers worked in pairs, one reading numbers to the other. At the west end of the viaduct where traffic was moving at a high speed, it was necessary to assign observers to each lane of traffic. In the time study of

traffic on the viaduct, lack of sufficient space at station 5 made it impossible to record numbers at the point of observation, and observers reading numbers communicated by means of portable telephones with recorders stationed across the roadway.

Recorders listing the license numbers were supplied with small cards, upon which were stamped the station number, the date and the time of day to the nearest minute. One card was used for each minute, and only the license numbers of vehicles passing during that minute were listed on a single card. At the beginning of each day's work, all timekeepers' watches were compared and set, and at frequent intervals during the day they were checked against a watch held by the general supervisor.

In classifying trucks, it was impossible to draw a sharp distinction between light trucks and heavy trucks. Observers were instructed to classify trucks of 2½-ton capacity or less as light trucks and all others as heavy, and reliance was placed upon their experience to make this distinction with reasonable accuracy. The classification of individual trucks at entrance and exit stations was found to be fairly consistent.

FIELD METHODS RESULTED IN SATISFACTORY SAMPLES

Theoretically, if the license numbers of half the vehicles passing are listed at random at each end of a given section of highway, one quarter of the cars can be identified and the travel time determined. In general terms, the percentage of cars identified at both ends will be the product of the percentages of the licenses listed at each end divided by 100. It is obvious that the number of probable matches decreases very rapidly as the efficiency in recording numbers is decreased. A further loss occurs when there are other entrances and exits to the route, not covered by recording stations, as was the case particularly on the old route. Therefore, every effort was made to maintain a high efficiency in listing license numbers. After a short period of training a surprising speed in recording was developed. In heavy traffic the number recorded per minute on a single card consistently exceeded 20 and occasionally exceeded 30.

The field study of the old route before the opening of the viaduct was made on September 24, 25, 26, 27, 28, and October 1 and 2, 1932. These dates included 2 Saturdays, 2 Sundays, and 3 week days. Eight hours of observation were made each day, staggered from day to day to cover the hours of the day from 6 a.m. to 10 p.m. When this schedule was completed, an examination of the data revealed that the volume of traffic and the number of bridge openings during many of the hours of observation duplicated those of other hours. To avoid unnecessary clerical work, subsequent analysis was confined to a selected sample of 19 hours of east-bound and 18 hours of west-bound traffic covering a complete range in volume of traffic and bridge openings.

Tabulation of this sample showed approximately 99,000 license numbers recorded at the eight stations, 85,000 being for passenger cars. During the same hours, the traffic passing these stations totaled 128,000 vehicles, including 112,500 passenger cars. Of the license numbers listed 37,000 were matched, thus giving the trip time of 18,500 vehicles. On the average 77 percent of the passing vehicles were listed, and 29 percent of these vehicles were timed over the route. Even under the difficult conditions prevailing after nightfall the efficiency averaged 24 percent. Under the severest test, heavy east-bound passenger-car traffic on Sunday night, a record of 17 percent was maintained.

Because a very considerable proportion of the vehicles passing through Tonnelle Circle (stations 4 and 5) entered or left the route at Newark Avenue, where there were no observation stations, a fairer measure of the efficiency of the method is to compare the number of timed vehicles with the total traffic entering the west end of Communipaw Avenue through stations 7 and 8 and leaving through stations 1 and 2. On this basis the average efficiency for the entire study was 38 percent.

The High-Level Viaduct was opened to traffic on November 23, 1932. In order to allow traffic to adjust itself to driving conditions on the viaduct and also to obtain a wide range in volume of traffic, observations of trip time were not made until Saturday and Sunday, May 20 and 21, 1933. The method of observation was the same as that described for the old route. Stations 1 and 8 were located at approximately the same points as in the previous study. Stations 4 and 5 were placed at the junction of the viaduct and the two ramps leading to and from Tonnelle Circle. This point was about 500 feet west of the previous location, a move so slight that its effect on the comparative trip time could be safely ignored. No additional observations of trip time on the old route were made, as the previous observations were sufficient to estimate the trip time under the conditions existing on the old route after the viaduct was opened.

TYPICAL CONGESTION ON NEWARK AVENUE AT INTERSECTION WITH U.S. NO. 1 BEFORE OPENING OF VIADUCT.

The number of vehicles entering or leaving the viaduct between the stations was less than on the old route and the proportion of cars timed was therefore higher. During 11 hours of observation 29,700 license numbers were recorded at the 4 stations out of a possible 34,000. Of the numbers listed 16,800 were matched, giving the trip time of 8,400 vehicles. On the average 87 percent of the passing vehicles were listed, and 49 percent were timed.

The large number of licenses listed during the observations on the old route and the variations in trip time caused by the bridge openings would have made the matching of numbers by inspection an almost impossible task. This work was greatly facilitated by the use of sorting and tabulating machines, the field record being transferred to punch cards and the matching accomplished mechanically. The license numbers listed in the time study on the viaduct were matched by inspec-

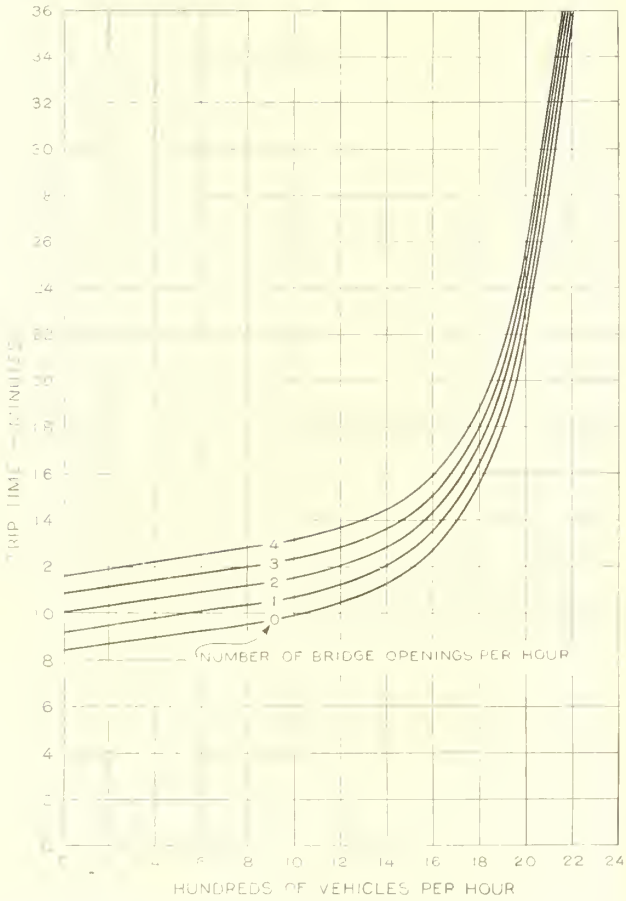


FIGURE 2.—VARIATION IN THE TRIP TIME OF PASSENGER CARS ON THE GROUND-LEVEL ROUTE BETWEEN TONNELLE CIRCLE AND THE WEST END OF THE VIADUCT, WITH VOLUME OF TRAFFIC AND FREQUENCY OF BRIDGE OPENINGS.

tion as the number of them was much smaller with little variation in individual trip times.

DEFINITE RELATION FOUND BETWEEN TRIP TIME, VOLUME OF TRAFFIC AND BRIDGE OPENINGS

Data from the field records as analyzed and tabulated consisted, for each selected period of time and for each desired combination of stations, of a chronological listing of vehicles according to time of arrival at the entering station together with trip time to the leaving station. The fluctuation in trip time from minute to minute could be observed and compared to variation in traffic volume, and also, on the old route, to the number of bridge openings. This tabulation was divided into hourly periods arranged to give the greatest possible variation in traffic volume and the number of bridge openings, and the trip times in each hour period were averaged. The average times were then plotted against traffic volume and bridge openings during the corresponding periods.

The traffic volume at the Hackensack River Bridge was used in estimating the average daily traffic on the old route. The openings of the two bridges were combined, the assumption being that the interruption to traffic was equivalent to that of one bridge. The graph indicated a definite relationship between trip time, traffic volume, and bridge openings, and a series of curves was fitted to the data by means of a multiple correlation to establish this relationship mathematically. Similar curves were fitted to the data for trip time on the viaduct.

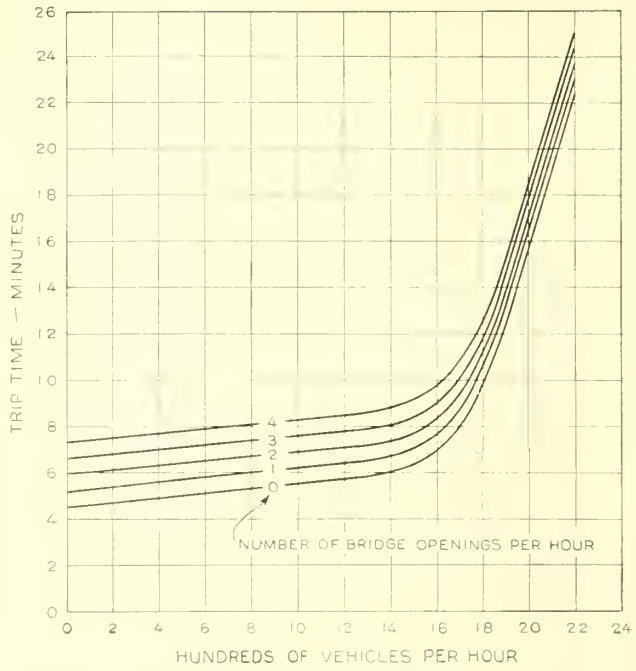


FIGURE 3.—VARIATION IN THE TRIP TIME OF PASSENGER CARS ON THE GROUND-LEVEL ROUTE BETWEEN WEST SIDE PARK AND THE WEST END OF THE VIADUCT, WITH VOLUME OF TRAFFIC AND FREQUENCY OF BRIDGE OPENINGS.

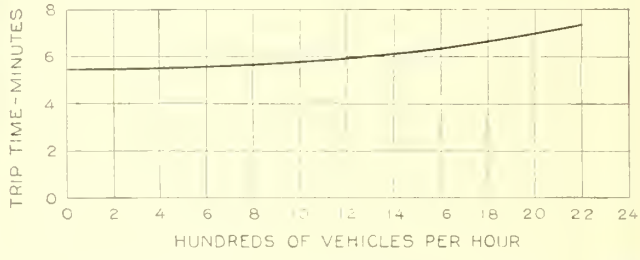
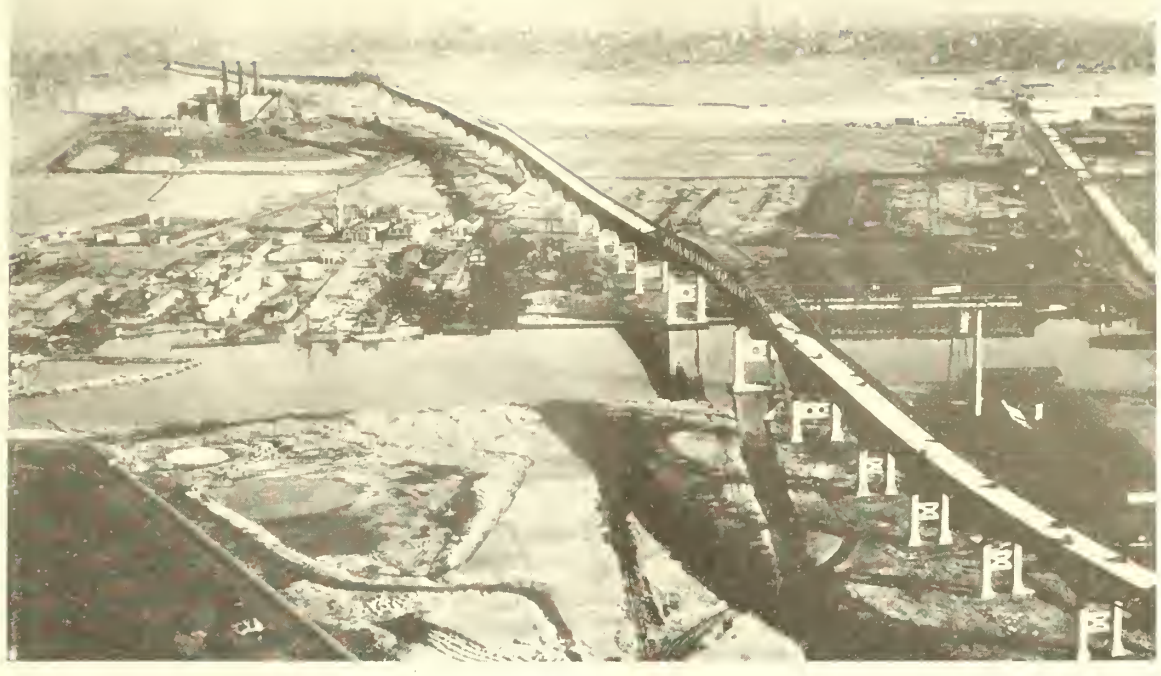


FIGURE 4.—VARIATION IN THE TRIP TIME OF PASSENGER CARS ON THE VIADUCT, WITH VOLUME OF TRAFFIC.

These curves showed the effect of the volume of traffic on the old route as measured at the Hackensack River Bridge and the combined number of bridge openings per hour at both bridges on the normal trip time for passenger cars, light trucks, and heavy trucks between the west end of the viaduct and Tonnelle Circle, and also for the same classes of vehicles along Communipaw Avenue. Figures 2, 3, and 4 show curves for passenger cars only on the old route and on the viaduct. The curves give the normal trip time on the old route for any combination of volume of traffic per hour from 0 to 2,200, and from 0 to 4 bridge openings per hour. They show the normal trip time on the viaduct for any volume of traffic from 0 to 2,200 vehicles per hour. This range in traffic volume and rate of bridge openings covered all observed fluctuations.

In order to establish the weighted average trip time throughout the day it was necessary to estimate accurately the volume of traffic during each hour of the day for the various classes of vehicles in each direction, and also, for traffic on the old route, the average number of bridge openings for every hour of the day. Knowing this, the trip time for every hour of the day for any type of vehicle could be determined from the curves. The average trip time for each type of vehicle was obtained by multiplying trip time for each hour and for each direction by the number of vehicles for each



GENERAL VIEW OF VIADUCT. THE COMMUNIPAW AVENUE ROUTE CAN BE SEEN AT THE RIGHT.

corresponding hour and direction, totaling these products and dividing by the total number of vehicles of the given type for the day. Weighted average trip times of passenger cars, light trucks, and heavy trucks were thus obtained for week days, Saturdays, and Sundays and holidays. The trip time of busses was originally recorded separately, but the number of bus trips was insufficient to establish a definite relationship with traffic or bridge openings. As the busses constituted a small percentage of total traffic, they were considered as passenger cars in this analysis. The trip time of busses appears to approximate very closely that of passenger cars, and if it is desired to estimate the time savings to busses, the trip-time curves of passenger cars may be used.

It was evident that in order to establish reliable averages for traffic during each hour of the day careful estimates were necessary. Numerous traffic counts made by the State Highway Department were available for the year 1932. These records consisted largely of counts at the Hackensack River Bridge, made in 8-hour shifts, and gave the volume of traffic per hour in either direction, classified as passenger cars, light trucks, and heavy trucks. In numerous cases shifts were arranged to give a continuous 16-hour count and for several consecutive days 24-hour counts were available. From these counts it was possible to compute the normal percentage of total daily traffic in each 8-hour shift, and to raise all counts to their 24-hour equivalents.

TRAFFIC COUNTS CORRECTED FOR SEASONAL VARIATION BY USE OF HOLLAND TUNNEL RECORDS

The first step in analyzing the available traffic records was to estimate the average daily traffic eastbound and west-bound for week days, Saturdays, and Sundays and holidays. While the traffic records were large in number the counts had not been made on a schedule such that a simple average of counts would give proper weight to the seasonal variations in traffic. However, complete and detailed traffic records for every day in the year at the Holland Tunnel were made available through the courtesy of the Port of New York Authority. A comparison of the individual traffic counts at the Hackensack River Bridge (raised to a 24-hour basis) with Holland Tunnel records for the same days, established that there was a constant ratio between the traffic volumes at the two points and that the seasonal fluctuations were similar. The average ratios of Hackensack River Bridge counts to Holland Tunnel counts were computed for both directions on week days, Saturdays, and Sundays and holidays. These average ratios were then applied to the 24-hour counts at the Holland Tunnel for the corresponding days (obtained from the complete year's record) to estimate the average daily traffic at the Hackensack River Bridge as shown in table 1.

While large variations occurred in the total daily traffic in various months of the year, the percentage distribution through the hours of the day remained constant. It was possible to compute from these records a reliable index showing the percentage of total daily traffic traveling during each hour of the day for week days, Saturdays, and Sundays and holidays for both east-bound and west-bound traffic. These indices when applied to the daily traffic gave estimates of the average hourly traffic of all classes of vehicles.

The New Jersey State Highway Commission maintains accurate records of the operation of all draw bridges under its jurisdiction, which show the time of occurrence and the duration of every draw opening. An examination of the records for the Hackensack and Passaic River Bridges for 1932 showed that openings of 5 minutes' duration greatly predominated, regard-

TABLE 1.—*Estimates of average daily traffic at the Hackensack River Bridge during 1932*

Day and direction	Average daily traffic, Holland Tunnel	Ratio Hackensack River Bridge traffic to Holland Tunnel traffic	Estimated average daily traffic at Hackensack River Bridge
Week day:			
East-bound	14,339	1.314	18,800
West-bound	14,175	1.312	18,600
Total	28,514	1.312	37,400
Saturday:			
East-bound	15,891	1.244	19,800
West-bound	17,467	1.206	21,000
Total	33,357	1.222	40,800
Sundays and holidays:			
East-bound	20,747	1.191	24,700
West-bound	19,821	1.240	24,600
Total	40,568	1.214	49,300

Total yearly traffic:	*Vehicles*
Holland Tunnel (recorded)	11,433,847
Hackensack River Bridge (estimated)	14,600,000

less of time of day, and the average duration of all openings was very nearly five minutes. Accordingly, the average opening was assumed to be 5 minutes, and computations were based on the number of openings rather than their duration. As previously explained, the two bridges were treated as a single bridge. The combined average number of openings for every hour of the day was computed for week days (including Saturdays), and for Sundays and holidays. Saturdays were included with week days, as openings on Saturdays were found to correspond closely with those of other week days. According to the New Jersey State Highway Commission, the total number of bridge openings of these two bridges during 1932 was less than in any other year since 1926 and was only one half the number during 1929. There is every indication of an increasing number of openings in future years, and consequently more frequent interruptions to traffic.

LARGE VOLUME OF TRAFFIC DIVERTED TO VIADUCT

These data in conjunction with the estimates of the hourly volume of traffic were used to estimate the various average trip times during every hour of the day, and from these the weighted average trip times. Table 2 shows the steps in the computation of the weighted average trip time on the old route between the west end of the viaduct and Tonnelle Circle for east-bound passenger cars on a week day. Column 1 of the table is the estimate of total hourly traffic at the Hackensack River Bridge, while column 2 is the average number of bridge openings per hour. The use of the figures in these two columns in conjunction with the trip-time curves, figure 2, results in the average trip time for each hour of the day, column 3. Car-minutes are obtained by multiplying the trip time for each hour by the number of passenger cars per hour, column 4. The weighted average trip time for the day is total car-minutes divided by total cars, which in this case is 12.1 minutes.

Having established an accurate estimate of average daily traffic at the Hackensack River Bridge it was necessary to estimate the division of traffic at the intersection of Communipaw Avenue and the highway to Tonnelle Circle near West Side Park. Only scattering

TABLE 2.—*Computation of weighted average trip time for week-day east-bound passenger cars traveling on the ground-level route between the west end of the viaduct and Tonnelle Circle*

Hours	(1) Total traffic Hackensack Bridge	(2) Number of bridge openings per hour	(3) Trip time of passenger cars, minutes	(4) Number of passenger cars	(3)×(4) Car-minutes per hour
12 to 1 a.m.	420	0.95	9.7	364	3,531
1 to 2 a.m.	280	.95	9.6	233	2,237
2 to 3 a.m.	150	.82	9.2	108	994
3 to 4 a.m.	140	.74	9.1	90	819
4 to 5 a.m.	150	.97	9.4	84	790
5 to 6 a.m.	200	1.39	9.9	101	1,000
6 to 7 a.m.	460	2.20	11.0	263	2,893
7 to 8 a.m.	870	2.91	12.1	572	6,921
8 to 9 a.m.	1,120	2.92	12.5	798	9,975
9 to 10 a.m.	1,240	3.09	13.1	908	11,895
10 to 11 a.m.	1,210	3.25	13.1	897	11,751
11 to 12 a.m.	1,130	3.14	12.8	829	10,611
12 to 1 p.m.	980	3.60	12.7	715	9,080
1 to 2 p.m.	980	3.27	12.5	710	8,875
2 to 3 p.m.	920	3.01	12.2	680	8,296
3 to 4 p.m.	1,100	2.76	12.3	849	10,443
4 to 5 p.m.	1,380	2.73	13.3	1,138	15,135
5 to 6 p.m.	1,430	2.43	13.3	1,207	16,053
6 to 7 p.m.	1,130	2.37	12.1	956	11,568
7 to 8 p.m.	1,010	1.78	11.4	844	9,622
8 to 9 p.m.	850	1.53	10.9	706	7,695
9 to 10 p.m.	730	1.22	10.5	598	6,279
10 to 11 p.m.	480	1.44	10.4	437	4,545
11 to 12 p.m.	440	0.86	9.6	398	3,821
Total	18,800			14,485	174,829

Weighted average trip time = 174,829 ÷ 14,485 = 12.1 minutes.

counts were available at this point, but they were sufficient to establish a reliable percentage.

The number of traffic counts at the Hackensack River Bridge after the opening of the viaduct was insufficient to set up accurate ratios with Holland Tunnel traffic. However, using the percentage distribution of traffic through the hours of the day as determined from the previous records, it was possible to raise the traffic counts to estimated 24-hour traffic, and to correct this estimate for seasonal fluctuations by means of the seasonal index for Holland Tunnel traffic. In no case did these corrected individual estimates differ materially from each other and it was felt that an average of them would indicate the daily traffic accurately. Table 3 shows these estimates of traffic before and after the opening of the viaduct. Estimates are given of the volume of traffic between Tonnelle Circle and the west end of the viaduct, and between West Side Park and the west end of the viaduct along Communipaw Avenue. The total of these equals the estimated traffic at the Hackensack River Bridge shown at the right of the table. The difference in these two estimates, that is, the reduction in traffic volume on the old route after the opening of the viaduct, appears at the bottom of the table.

VIADUCT RESULTS IN LARGE SAVING IN VEHICLE TIME

The great difference in the volume of traffic traveling between Tonnelle Circle and the west end of the viaduct, before and after the opening of the viaduct, may be considered as a measure of the traffic which has been diverted to the viaduct, and which unquestionably benefits by the saving in time between these points. Table 3, however, shows that this difference in traffic to and from Tonnelle Circle accounts for only a part of the total reduction in traffic on Communipaw Avenue at the Hackensack River Bridge. This may be explained by the fact that previous to the opening of the viaduct, traffic destined to the George Washington Bridge and towns north of Tonnelle Circle was routed

TABLE 3.—*Daily traffic at the Hackensack River Bridge and the amount of this traffic entering and leaving the route at Tonnelle Circle and at the West Side Park*

DAILY TRAFFIC BEFORE THE OPENING OF THE VIADUCT

	To and from Tonnelle Circle				To and from West Side Park				Total at the Hackensack River Bridge			
	Passenger cars	Light trucks	Heavy trucks	Total	Passenger cars	Light trucks	Heavy truck	Total	Passenger cars	Light trucks	Heavy trucks	Total
Week day	14,485	1,625	1,950	18,060	14,485	2,155	2,700	19,340	28,970	3,780	4,650	37,400
Saturday	23,212	1,193	1,352	25,757	12,498	1,192	1,353	15,043	35,710	2,385	2,705	10,800
Sundays and holidays	30,596	688	126	31,710	16,474	689	427	17,590	47,070	1,377	853	49,300

DAILY TRAFFIC AFTER THE OPENING OF THE VIADUCT

Week day	790	376	634	1,800	10,500	1,600	2,700	14,800	11,290	1,976	3,334	16,600
Saturday	1,936	236	641	2,813	11,894	910	1,353	14,187	13,830	1,176	1,991	17,060
Sundays and holidays	796	18	16	830	9,144	334	292	9,770	9,940	352	308	10,600

DAILY TRAFFIC DIVERTED TO THE VIADUCT

Week day	13,695	1,249	1,316	16,260	3,985	555	0	4,540	17,680	1,804	1,316	20,800
Saturday	21,276	957	711	22,944	604	252	0	856	21,880	1,209	711	23,800
Sundays and holidays	29,800	670	410	30,880	7,330	355	135	7,820	37,130	1,025	545	38,700

through West Side Park to Hudson Boulevard to reduce congestion at Tonnelle Circle. All such traffic is undoubtedly now using the viaduct, and it is reasonable to assume that it benefits by a saving in trip time at least as great as that of the traffic which was routed through Tonnelle Circle.

Accordingly, two estimates have been prepared for the total vehicular time saved by the viaduct. The first of these includes only savings to those vehicles which previously traveled between Tonnelle Circle and the west end of the viaduct, and is termed the minimum estimate. The second, or maximum estimate, is based on the total reduction of traffic at the Hackensack River Bridge, with the assumption that all of these vehicles have been diverted to the viaduct and benefit by a saving in trip time at least equal to that of the traffic previously routed through Tonnelle Circle.

Table 4 is a comparison of the weighted average trip time of the various classes of vehicles before and after the opening of the viaduct. It is evident that in addition to a considerable time saving to those vehicles which now use the viaduct, there is also a noticeable time saving per trip between various points on the old route because of the lessened volume of traffic now existing.

Table 5 is an estimate of the total yearly saving in vehicle minutes to this traffic. The average daily traffic multiplied by the saving per trip in minutes gives the average vehicle minutes saved per day and this multiplied by the number of days per year gives the total saving per year in vehicle minutes. For example, table 3 shows that on week days 790 passenger cars per day now travel on the old route between Tonnelle Circle and the west end of the viaduct. From table 4 the weighted average trip time for vehicles making this trip before the opening of the viaduct was 12.0 minutes, and after the opening of the viaduct 11.0 minutes, a saving of 1.0 minute per trip. Seven hundred and ninety vehicles per day multiplied by 1.0 minute gives 790 vehicle-minutes saved each week day, multiplied by 254 gives an estimate of 201,000 vehicle-minutes per year. The total yearly saving for all types of vehicles resulting from reduced traffic congestion on the old route is 5,542,000 vehicle-minutes per year.

TABLE 4.—*Summary of average trip times in minutes*

ON GROUND-LEVEL ROUTE BETWEEN WEST SIDE PARK AND WEST END OF VIADUCT

	Passenger cars			Light trucks			Heavy trucks		
	Before opening viaduct	After opening viaduct	Saving in time	Before opening viaduct	After opening viaduct	Saving in time	Before opening viaduct	After opening viaduct	Saving in time
Week days	7.2	6.6	0.6	8.2	7.5	0.7	8.9	8.2	0.7
Saturdays	7.3	6.6	.7	8.4	7.6	.8	9.2	8.4	.8
Sundays and holidays	8.7	5.1	3.3	7.9	6.0	1.9	10.5	6.7	3.8

ON GROUND-LEVEL ROUTE BETWEEN TONNELLE CIRCLE AND WEST END OF VIADUCT

Week days	12.0	11.0	1.0	13.9	13.0	0.9	14.9	14.3	0.6
Saturdays	12.2	11.0	1.2	14.2	13.0	1.2	15.3	14.5	.8
Sundays and holidays	14.0	9.5	4.5	13.9	11.0	2.9	18.5	12.4	6.1

ON GROUND-LEVEL ROUTE AND ON VIADUCT BETWEEN TONNELLE CIRCLE AND WEST END OF VIADUCT

Week days	12.0	[2]5.7	6.3	[1]13.9	[2]6.4	7.5	[1]14.9	[2]7.5	7.4
Saturdays	12.2	[2]5.8	6.4	[1]14.2	[2]6.4	7.8	[1]15.3	[2]7.6	7.7
Sundays and holidays	[1]14.0	[2]6.0	8.0	[1]13.9	[2]6.6	7.3	[1]18.5	[2]8.0	10.5

[1] On ground-level route.
[2] On viaduct.

Table 6 is an estimate of the yearly saving prepared in the same manner for the vehicles which previously traveled on the old route between Tonnelle Circle and the west end of the viaduct and which now use the viaduct. This estimate, which has been termed the minimum estimate, is 49,130,000 vehicle-minutes per year. To this is added the saving to vehicles still using the ground-level route, the final minimum estimate being 54,672,000 vehicle-minutes saved per year.

Table 7 is an estimate of the yearly saving if the saving per trip is assumed to apply to the total reduction in traffic on the old route, the maximum estimate. This estimate is 60,563,000 vehicle-minutes per year, or 66,105,000 when the saving to vehicles still using the ground-level route is added. In all of these tables the total estimated saving is a sum of separate estimates for passenger cars, light trucks, and heavy trucks, and

TABLE 5.—*Estimate of time saved per year by vehicles using the ground-level route after the opening of the viaduct*

BETWEEN TONNELLE CIRCLE AND THE WEST END OF THE VIADUCT

	Daily traffic	Minutes saved per trip	Vehicle-minutes saved		
			Per day	Number of days	Per year
Week day:					
Passenger cars	790	1.0	790	254	201,000
Light trucks	376	.9	338	254	86,000
Heavy trucks	634	.6	380	254	97,000
Saturday:					
Passenger cars	1,936	1.2	2,323	52	121,000
Light trucks	236	1.2	283	52	15,000
Heavy trucks	641	.8	513	52	27,000
Sundays and holidays:					
Passenger cars	796	4.5	3,582	59	211,000
Light trucks	18	2.9	52	59	3,000
Heavy trucks	16	6.1	98	59	6,000

BETWEEN WEST SIDE PARK AND THE WEST END OF THE VIADUCT

Week day:					
Passenger cars	10,500	0.6	6,300	254	1,600,000
Light trucks	1,600	.7	1,120	254	284,000
Heavy trucks	2,700	.7	1,890	254	480,000
Saturday:					
Passenger cars	11,894	.7	8,326	52	433,000
Light trucks	940	.8	752	52	39,000
Heavy trucks	1,353	.8	1,082	52	56,000
Sundays and holidays					
Passenger cars	9,144	3.3	30,175	59	1,780,000
Light trucks	334	1.9	635	59	38,000
Heavy trucks	292	3.8	1,110	59	65,000

Totol vehicle-minutes saved per year

Passenger cars	4,346,000
Light trucks	465,000
Heavy trucks	731,000
Total	5,542,000

these estimates are shown in detail. The yearly savings for the various classes of vehicles for the minimum and maximum estimates are summarized in table 8.

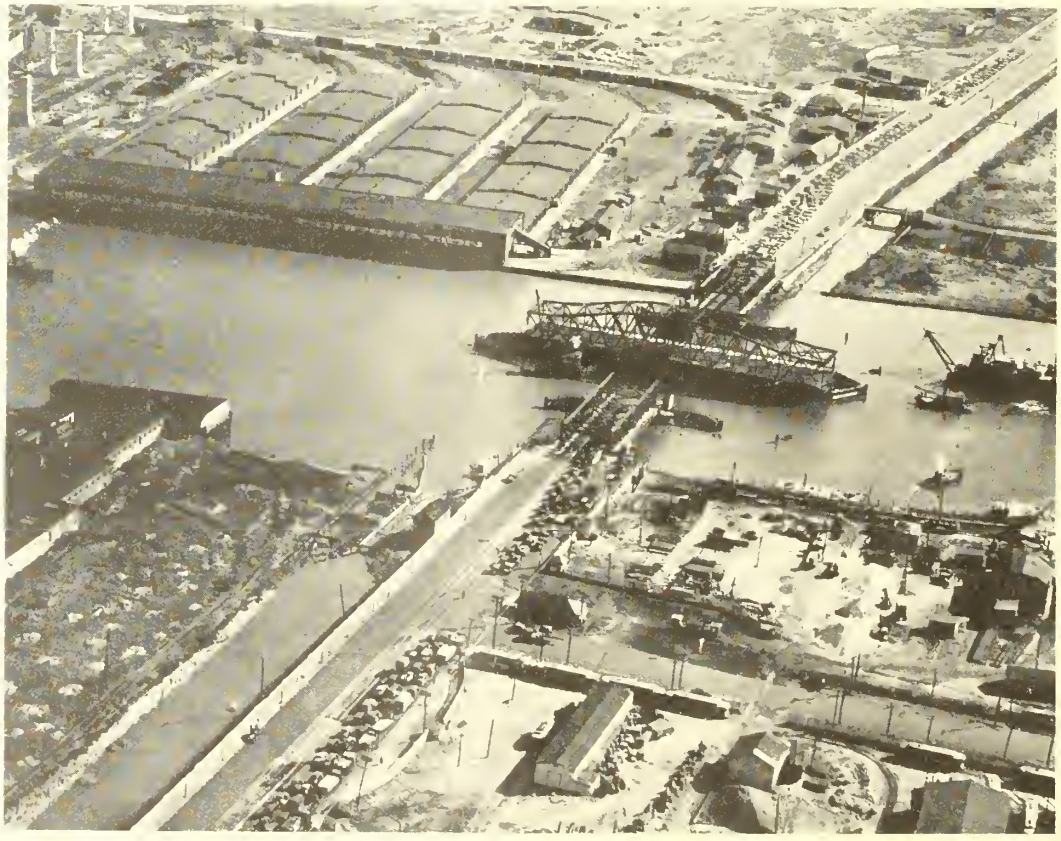
The construction cost of the viaduct was approximately $19,300,000. In the original economic study by the State authorities it was assumed that the money value of the prospective savings could be capitalized

TABLE 6.—*Minimum estimate of time saved per year by vehicles which previously traveled on the ground-level route between Tonnelle Circle and the west end of the viaduct*

	Daily vehicles	Minutes saved per trip	Vehicle-minutes saved		
			Per day	Number of days	Per year
Week day:					
Passenger cars	13,695	6.3	86,278	254	21,915,000
Light trucks	1,249	7.5	9,368	254	2,379,000
Heavy trucks	1,316	7.4	9,738	254	2,474,000
Saturday:					
Passenger cars	21,276	6.4	136,166	52	7,081,000
Light trucks	957	7.8	7,465	52	388,000
Heavy trucks	711	7.7	5,475	52	285,000
Sundays and holidays:					
Passenger cars	29,800	8.0	238,400	59	14,065,000
Light trucks	670	7.3	4,891	59	289,000
Heavy trucks	410	10.5	4,305	59	254,000

Total vehicle-minutes saved per year

	Passenger cars	Light trucks	Heavy trucks	Total
On the viaduct	43,061,000	3,056,000	3,013,000	49,130,000
On the ground-level route	4,346,000	465,000	731,000	5,542,000
Total	47,407,000	3,521,000	3,744,000	54,672,000



TRAFFIC DELAYED ON COMMUNIPAW AVENUE AS A RESULT OF BRIDGE OPENING.

TABLE 7.—*Maximum estimate of time saved per year by total vehicles diverted to the viaduct from Communipaw Avenue*

	Daily vehicles	Minutes saved per trip	Vehicle-minutes saved		
			Per day	Number of days	Per year
Week day:					
Passenger cars	17,680	6.3	111,384	254	28,292,000
Light trucks	1,804	7.5	13,530	254	3,437,000
Heavy trucks	1,316	7.4	9,738	254	2,473,000
Saturday:					
Passenger cars	21,880	6.4	140,032	52	7,282,000
Light trucks	1,209	7.8	9,430	52	490,000
Heavy trucks	711	7.7	5,475	52	285,000
Sundays and holidays:					
Passenger cars	37,130	8.0	297,040	59	17,525,000
Light trucks	1,025	7.3	7,482	59	441,000
Heavy trucks	545	10.5	5,722	59	338,000

Total vehicle-minutes saved per year

	Passenger cars	Light trucks	Heavy trucks	Total
On the viaduct	53,099,000	4,368,000	3,096,000	60,563,000
On the ground-level route	4,346,000	465,000	731,000	5,542,000
Total	57,445,000	4,833,000	3,827,000	66,105,000

TABLE 8.—*Summary of vehicle-minutes saved per year by type of vehicle*

	Minimum estimate	Maximum estimate
Passenger cars	47,407,000	57,445,000
Light trucks	3,521,000	4,833,000
Heavy trucks	3,744,000	3,827,000
Total	54,672,000	66,105,000

at 6 percent to obtain the sum which might justifiably be spent for the contemplated improvements. Six percent of the actual construction cost is $1,158,000, and in order to justify the construction of the viaduct on the basis of the above minimum and maximum estimates a vehicle-minute must be valued at 2.12 or 1.75 cents.

CONSTRUCTION OF VIADUCT ADVANTAGEOUS TO TRAFFIC ON OTHER ROUTES

It is emphasized that the above estimates of time saved do not include all the savings which may be credited to the viaduct. No estimate of the value of shortened travel distance has been made, and the traffic on the viaduct to which the average saving in trip time was applied is not the total volume of traffic actually using the viaduct. For the maximum estimate the volume used was 20,800 per day for week days, 23,800 per day for Saturdays, and 38,700 per day for Sundays and holidays. From the most recent counts made on the viaduct it is probable that the actual traffic for these days is 30,000, 35,000, and 45,000 vehicles per day. The viaduct undoubtedly has attracted a considerable volume of traffic which previously avoided the route entirely because of the congestion, and while no estimate is possible of the saving in time enjoyed by this additional traffic, it must be considerable. There is also every reason to believe that the traffic on the viaduct will increase during the next few years to a volume which could not have been adequately served by the old route.

Neither the gross figures of estimated savings nor the saving in the average time per trip gives the complete picture of traffic congestion on the old route. The intensity of this congestion during hours of peak traffic is illustrated by the time study on Sunday evening, October 2. From 6:30 p.m. on that day to 9.00 p.m. east-bound traffic toward New York on Communipaw Avenue exceeded 1,800 vehicles per hour and reached a maximum of 2,100, while the bridges were opened four times for a total of 20 minutes. The average trip time of passenger cars from the west end of the viaduct to Tonnelle Circle for the whole period was 29 minutes, while the trip along Communipaw Avenue required 19 minutes. During a comparable period on the viaduct, Sunday, May 21, 1933, the volume of east-bound traffic toward New York was equally heavy, but the average trip time of passenger cars was only 6.7 minutes, and traffic congestion was not noticeable.

The curves for average trip time of passenger cars on the old route in figure 2, should be compared with the same curve for the trip time on the viaduct, figure 4. On the old route, even without bridge openings, the curve for trip time rises very rapidly when the traffic volume exceeds 1,300 vehicles per hour, indicating congestion beyond that point. On the viaduct, however, the curve of trip time does not show any such marked increase up to 2,200 vehicles per hour, the maximum traffic observed. On the old route during the average week day, traffic exceeded 1,300 vehicles per hour in one direction or both during 2 hours of the day, on Saturdays during 5 hours, and on Sundays and holidays during 13 hours, and the average trip time of passenger cars during these hours was 13.2, 14.6, and 17.9 minutes, respectively. On the viaduct during these same hours the average trip time was 5.8, 6.0, and 6.3 minutes.

The volume of traffic on the old route has been so much reduced that even during the periods of heaviest traffic it does not exceed an average of 600 vehicles per hour in one direction on Communipaw Avenue and probably not more than 100 vehicles per hour on the section north from Communipaw Avenue to Newark Avenue. The result is much greater freedom of movement for local traffic, particularly at the previously congested intersections. The improvement of conditions at the junction where Newark Avenue crosses the old route is an illustration of such an advantage.

Newark Avenue (see fig. 1) is the principal artery to Jersey City and the Holland Tunnel for two important local routes from the west: the Belleville Turnpike, serving the suburban communities of Montclair, Glen Ridge, Bloomfield, and Belleville; and the Jersey City and Newark Turnpike, serving the northern section of Newark and the Oranges. In consequence, there is a considerable volume of traffic on Newark Avenue west of its intersection with the old route, only a comparatively small amount of which has been attracted to the viaduct. East of the intersection, the volume of traffic is considerably less, as a large part of the east-bound traffic turns left from Newark Avenue and proceeds through Tonnelle Circle, while the west-bound traffic is increased by a number of vehicles from Tonnelle Circle turning right into Newark Avenue. Previous to the opening of the viaduct, the large volume of east-bound traffic and the heavy left-turn movement, in conflict with an even larger flow of cross traffic were the cause of a great deal of congestion. Traffic was occasionally blocked in a standing line as far east as Tonnelle Circle, a distance of nearly one half mile, while

east-bound traffic on Newark Avenue not infrequently was blocked in two lanes for more than one quarter of a mile.

The transfer of through traffic to the viaduct has largely eliminated congestion at this intersection and has resulted in a material time saving to traffic using Newark Avenue. The relative magnitude of this saving was obtained from detailed studies of travel time on Newark Avenue during several hours of peak traffic in the first and second general time studies. An electric time-recording device was used to determine the travel time of vehicles to the nearest second over a distance which included all waiting and delay at the intersection. East-bound traffic was timed through the intersection from a point approximately 800 feet west, while west-bound traffic was timed from a point about 350 feet east of the intersection.

The time previously required for the average vehicle on Newark Avenue to travel these distances during hours of peak traffic was 74.9 seconds east-bound and 56.3 seconds west-bound. These averages are based on movement of 4,700 vehicles east-bound and 1,200 vehicles west-bound, and the average hourly flow during the observations was 800 and 300. After the viaduct was opened, 2,500 east-bound vehicles averaged 34.6 seconds over the same distance and 400 west-bound vehicles averaged 22.4 seconds. The corresponding hourly rates were 700 and 250. This indicates that for nearly the same volume on Newark Avenue there has been a reduction in the time required to pass through the intersection of 40.3 seconds per car or 53 percent for east-bound traffic, and 33.9 seconds or 60 percent for west-bound traffic. These results apply to only a limited period of observation, and no effort was made to calculate the saving on a yearly basis. It is clear, however, that it constitutes a very considerable item of incidental benefit resulting from the new viaduct construction.

RESULTS OF STUDIES ON ROGUE RIVER BRIDGE REPORTED

The final report on Application of Freyssinet Method of Concrete Arch Construction to the Rogue River Bridge in Oregon, by Albin L. Gemeny, of the Bureau of Public Roads, and C. B. McCullough, of the Oregon State Highway Commission, has been published as Bulletin No. 2 of the Oregon State Highway Commission.

In the Freyssinet method hydraulic jacks are placed in open radial joints appropriately located in arch ribs and used to produce deformations in the arch.

The deformations are controlled so as to produce the most desirable distribution of stress in the ribs, eliminating or mitigating the effects of volume changes in the concrete, foundation movements and the indeterminate effect of the spandrel structure. The jacks may be placed at one or more joints. For symmetrical arches a joint at the crown is usually sufficient. After the arch has been adjusted to the desired position, the joint is filled and the jacks removed, after which the structure acts as the usual type of fixed arch. If there is reason to believe that future adjustments may be necessary the jack emplacements may be preserved for this purpose.

The Rogue River Bridge is a Federal-aid project constructed jointly by the Bureau of Public Roads and the Oregon State Highway Commission. It consists of seven 230-foot spans with a 3-span central group on elastic piers and a 2-span group at each end.

During the jacking operations careful measurements were made of deformations and strains which occurred in the arches so that, at all times, the effect of the jacking was known. The report contains these data in the form of charts and tables.

Satisfactory results were obtained with the Freyssinet method in economy of bridge construction and also from the standpoint of certainty of stress action.

The Bureau of Public Roads has available a limited supply of the report for distribution to libraries, highway bridge engineers, and instructors in bridge design.

THE EFFECT OF CONTROL METHODS ON TRAFFIC FLOW

Reported by E. H. HOLMES, Assistant Highway Economist, Division of Highway Transport, United States Bureau of Public Roads

FIGURE 1.—INTERSECTION OF CONSTITUTION AVENUE AND SEVENTEENTH STREET. TRAFFIC MOVING ON CONSTITUTION AVENUE.

STREET and highway intersections not only determine the capacity of the interseeting ways, but also may cause serious delay to traffic. The intersection of Constitution Avenue and Seventeenth Street in Washington, D.C. (fig. 1), offered unusual opportunity to analyze the delay eaused by different control methods to a moderately heavy flow of traffic. This is but one of a series of intersection studies now being eonducted by the Bureau of Public Roads, but the methods used in the eollection of the data and the results obtained seem to justify a special report on this partieular study.

VOLUME OF TRAFFIC NEARLY CONSTANT DURING PERIOD OF STUDY

The steady flow of traffic, both passenger and commercial, a flexible eontrol mechanism and the absence of other controlled intersections in the vieinity made this an ideal intersection for such a study. In addition to vehicle-actuated control, the mechanism permitted fixed-time eontrol with intervals on either street ranging from 15 seeonds to 70 seconds, irrespective of the interval length on the other. Cooperation of the Park Police made possible the use of offieer eontrol, and also the study of traffie under no-control. The one-way street entering from the park (fig. 2) was shut off during the study, to provide a true four-way intersection.

The hours seleeted for the study were from 9:30 a.m. to 4:15 p.m. This period excluded the morning and afternoon rush hours and permitted the timing of a volume of traffic nearly constant at about 2,000 vehicles per hour. During this period the effects of left turns and of pedestrian movement, often considerable factors, were negligible. Rush hour traffic was neglected, beeause the intent of the study was to compare the time of vehicles under a constant volume with variable control methods, rather than to determine how the time might vary with volume; and because the rate of ehange of volume was so great during the rush hour that the flow would not remain eonstant over a long enough period to permit analysis of the lost time for any given volume.

In the passage of traffic through an intersection, time may be lost in stopping, in deceleration and acceleration, and possibly in slower than normal driving in anticipation of a signal change. The distanee within which these delays occur may be called the zone of influence of the interseetion and varies in length with the intersection, and with the speed of traffic. A graphic time recorder was used to record all vehicles as they entered and left the zone of influence, from which the average time per vehicle was determined as described in the Appendix.

During the course of observations at this intersection, earried on during the summer of 1932, the movement of traffic under no-control, officer control, vehicle-actuated control, and under 17 different fixed-time controls was studied. Each timing or control method was studied for a minimum of 2 to 3 hours, while certain control methods were observed for two or more days. The

time spent in actual collection of data amounted to about 56 hours, during which over 100,000 vehicles were timed and otherwise noted during their passage through the intersection.

Analysis of the time recorder charts showed the time of passage for the average vehicle entering on each of the four ways for each control method. These results are summarized in table 1, together with the average, weighted according to the number of vehicles, for each street and finally for the entire intersection. Since Constitution Avenue carried, roughly, twice the volume on Seventeenth Street, those control methods which gave a lower time for the Constitution Avenue traffic showed a correspondingly low figure for the average time for all vehicles.

TABLE 1.—*Average time of vehicle passage through zone of influence of intersection in seconds*

Timing		Constitution Avenue			Seventeenth Street			Both streets
Constitution Avenue	Seventeenth Street	West	East	Both directions	North	South	Both directions	
(1)	(1)	23.8	25.9	24.9	30.1	28.2	29.1	26.3
(2)	(2)	26.8	29.1	28.0	34.6	33.2	33.9	29.9
15	15	29.3	32.2	30.9	32.6	32.9	32.8	31.5
(3)	(3)	28.9	30.6	29.8	35.3	35.3	35.3	31.6
20	20	31.4	32.4	31.9	33.5	32.0	32.7	32.2
40	20	27.6	28.9	28.3	42.0	41.2	41.6	32.7
30	15	29.5	30.7	30.2	41.1	39.7	40.4	33.5
30	20	30.4	31.3	30.9	41.0	38.3	39.6	33.8
15	30	35.8	38.0	37.0	30.9	29.1	30.0	34.9
30	30	33.7	34.8	34.3	37.2	35.5	36.2	34.9
40	30	31.3	33.3	32.4	41.5	41.7	41.6	35.3
20	30	37.0	36.6	36.8	36.0	34.3	35.1	36.2
60	30	29.9	30.9	30.5	51.4	50.8	51.1	37.1
40	40	36.9	38.0	37.3	38.9	38.9	38.9	37.8
70	30	28.4	29.0	28.7	61.1	55.7	58.3	39.0
50	50	41.7	40.9	41.3	40.6	40.2	40.4	41.0
60	45	35.9	38.4	37.3	51.1	49.5	50.3	41.4
30	50	46.0	46.8	46.4	32.9	32.0	32.5	41.6
60	60	44.1	45.1	44.6	48.6	45.3	47.0	45.4
30	70	56.1	57.3	56.7	32.6	31.7	32.1	48.6
AVERAGE VEHICLES PER HOUR								
--------	--------	625	740	1,365	320	350	670	2,035

[1] No control.
[2] Officer control.
[3] Traffic-actuated control.

SHORT LIGHT INTERVALS BEST UNDER FIXED-TIME CONTROL

From the figures in table 1 there has been derived the following equation:

$$T = 26.4 + 0.04x + 0.28y \quad (1)$$

where T = time for the average vehicle to travel 600 feet, including the intersection.
x = green interval on Constitution Avenue (including 3-second amber overlap).
y = green interval on Seventeenth Street (including 3-second amber overlap).

The equation was obtained from a multiple correlation of three variables, one T, dependent, and the others, x and y, independent variables.

This equation shows that lengthening the light interval on either or both streets increases the average time of passage. It is to be expected that lengthening the interval on Seventeenth Street, which carries only one third of the traffic through the intersection, would increase the time of the average vehicle since stopping the heavy traffic to permit the light traffic to flow must necessarily result in an increased time per vehicle when the average is appropriately weighted. What is not expected, however, is that extending the interval on Constitution Avenue, the heavily traveled street, also results in an increased time for the average vehicle.

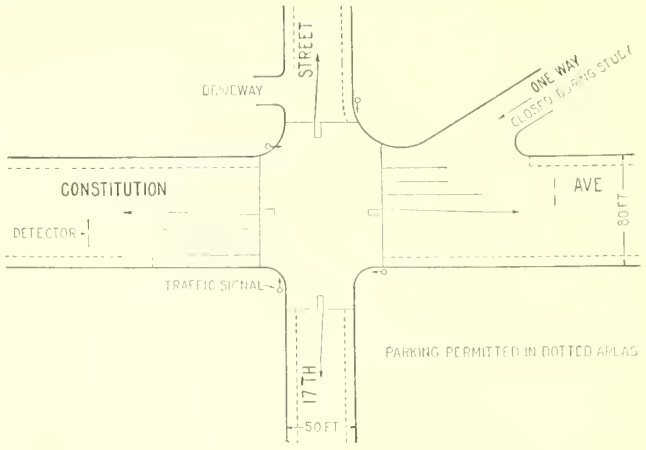


FIGURE 2.—PLAN OF INTERSECTION OF CONSTITUTION AVENUE AND SEVENTEENTH STREET.

Comparison of the actual time period for various light intervals with the time periods derived with equation 1 shows that through the middle of the range of light intervals studied the actual values are less than the theoretical, whereas in the upper and lower ranges the actual times are higher.

It is probable that decreasing the light interval below a certain figure would result in increasing the running time, and that inordinately long intervals increase the running time disproportionately. Figures 3, 4, and 5 show the effect of varying the light interval. Figure 3 shows the effect of increasing the cycle length, using the same cycle on both streets. With an interval length of 15 seconds (12 green plus 3 green and amber) on each street, or a total cycle length of 30 seconds, the average vehicle requires 31.5 seconds to traverse the zone of influence of the intersection. Maintaining this same 1 to 1 proportioning, a cycle of 120 seconds increases the running time to over 45 seconds per vehicle, an increase of nearly 50 percent.

NO ADVANTAGE IN FAVORING HEAVILY TRAVELED STREET WITH EXTREMELY LONG INTERVALS

Figure 4 shows the variation in the running time on each street as the green interval length on Constitution Avenue is varied while the green and amber interval on Seventeenth Street remains constant at 30 seconds. The average vehicle time on Constitution Avenue becomes less as the interval length is extended up to about 40 seconds, after which the rate of decrease in running time becomes increasingly less. Meanwhile the time has been increasing for the vehicles using Seventeenth Street, with the result that above a 40-second interval on Constitution Avenue, the time for the average vehicle begins to increase, even though the more heavily traveled street is the one favored by the control. Figure 6 shows accumulated traffic during a long light interval.

Figure 5 shows the effect of increasing the light interval on the lightly traveled street, while that on the heavily traveled street remains constant. Here the effect noticed in figure 4 is more marked. While the time of the average vehicle on Seventeenth Street decreases but slightly through the longer interval lengths, the time per vehicle on the heavily traveled street mounts very rapidly, and necessarily results in a very

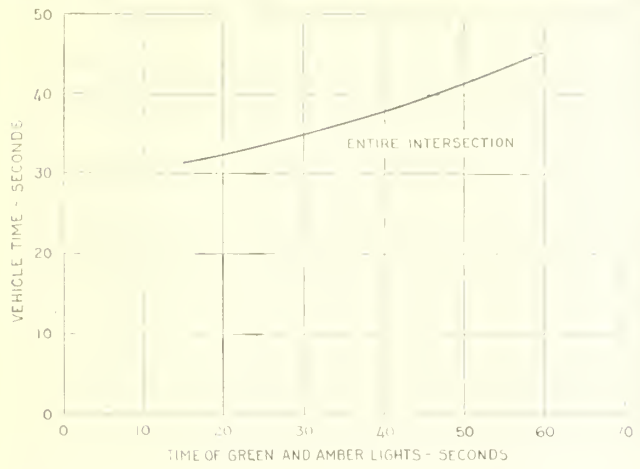


FIGURE 3.—RELATION BETWEEN LIGHT INTERVAL (SAME INTERVAL ON BOTH STREETS) AND AVERAGE TIME OF PASSAGE FOR BOTH STREETS.

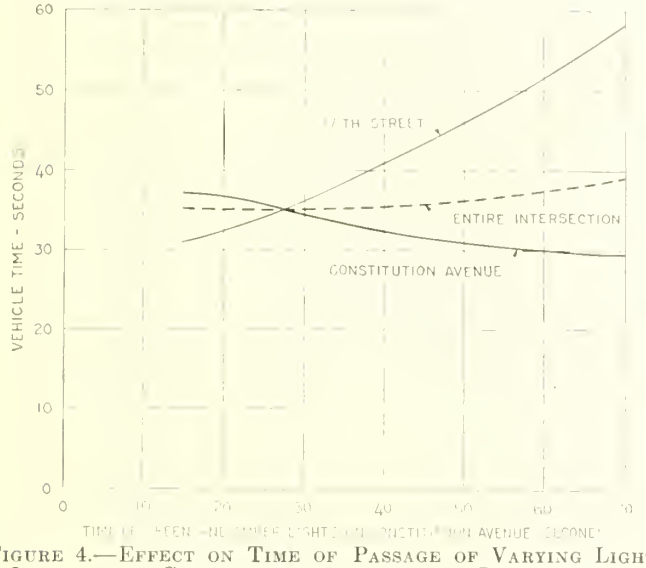


FIGURE 4.—EFFECT ON TIME OF PASSAGE OF VARYING LIGHT INTERVAL ON CONSTITUTION AVENUE WITH LIGHT INTERVAL ON SEVENTEENTH STREET KEPT CONSTANT AT 30 SECONDS.

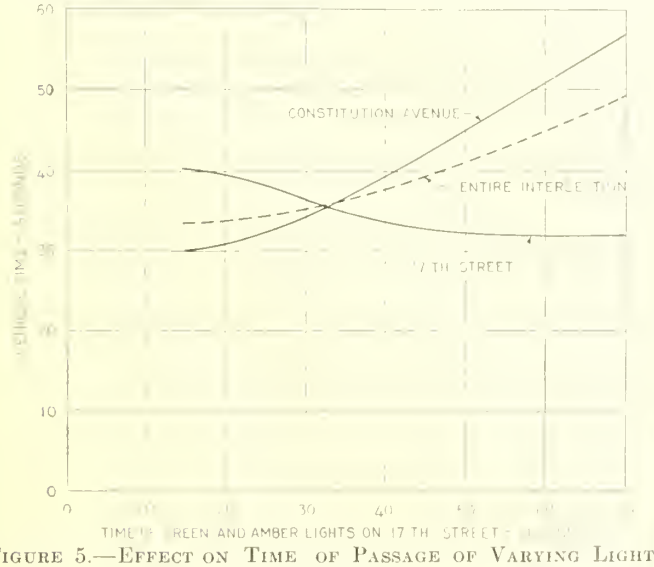


FIGURE 5.—EFFECT ON TIME OF PASSAGE OF VARYING LIGHT INTERVAL ON SEVENTEENTH STREET WITH LIGHT INTERVAL ON CONSTITUTION AVENUE KEPT CONSTANT AT 30 SECONDS.

rapid increase in the average time per vehicle for the entire intersection, when the times are weighted according to the volume on each street

These curves show very definitely that there is no advantage in favoring the heavily traveled street with extremely long intervals. When, following the appearance of the green light, the waiting vehicles have been cleared out and the retarding effect of the stopped vehicles on those immediately following them has disappeared, the flow of traffic through the intersection becomes the uninterrupted normal flow on the street. Further maintaining of the green interval does not result in a material saving of time to vehicles on this street, while the aggregate delay to those vehicles waiting on the other street is mounting more and more rapidly.

That unduly long cycle lengths are a hindrance to the free flow of traffic cannot be too strongly emphasized. Table 2 summarizes for easier comparison certain figures from table 1.

TABLE 2.—*Effect of cycle length on time of vehicle passage through zone of influence*

Green and amber interval		Cycle	Ratio, Constitution Avenue to Seventeenth Street	Average time per vehicle
Constitution Avenue	Seventeenth Street			
Seconds	*Seconds*	*Seconds*		*Seconds*
15	15	30	1 to 1	31.5
60	60	120	1 to 1	45.4
30	70	100	3 to 7	48.6
30	15	45	2 to 1	33.5
15	30	45	1 to 2	34.9
60	30	90	2 to 1	37.1

Table 2 reveals that the fastest movement under fixed time is attained with a setting of 15 seconds on each street, and that the slowest movement is when the control is set at 70 seconds on Seventeenth Street and 30 seconds on Constitution Avenue. This time, however, is but little higher than that for an interval of 60 seconds on each street. The 60—60 setting is at one extreme, while the 15—15 is at the other; the interval ratio is the same but the passage times are the lowest and practically the highest, respectively

With a signal setting of 30 seconds on Constitution Avenue and 15 seconds on Seventeenth Street, the time per vehicle is 33.5 seconds. Reversing the timing conditions on the two streets permits the average vehicle to traverse the zone of influence in 34.9 seconds, an increase of 1.4 seconds per vehicle. Using the same ratio but with a setting of 60 seconds on Constitution Avenue and 30 seconds on Seventeenth Street, the average vehicle time is 37.1 seconds, an increase of 3.6 seconds over the 30—15 setting. Lengthening the cycle in this case produced a greater increase in the time per vehicle than reversing the interval on the two streets while retaining the short cycle. At this intersection, it was more important that the cycle be kept short than that the time within the cycle be correctly proportioned. It is understood, of course, that this is not a general statement to include all conditions. Obviously it cannot be said that with congested traffic, the proportioning is immaterial, but it is strongly emphasized that under any conditions the cycle must be kept as short as practicable.

FIGURE 6.—LONG LIGHT INTERVALS UNDER FIXED CONTROL RESULTED IN WAITING LINES AND HIGH AVERAGE VEHICLE TIME.

FLEXIBLE CONTROLS PRODUCE RAPID VEHICLE MOVEMENT

The study of each control setting covered a period of at least 2 hours. This was considered satisfactory for the fixed times, since the movement was very regular, but more study was devoted to the flexible control methods to insure that no special condition might influence the flow of traffic, which by nature of the control would be somewhat variable. Accordingly, studies of officer control included some 7,200 vehicles, those made without control covered approximately 11,300 vehicles, and those under traffic-actuated control covered over 20,000 vehicles. Of the three flexible methods, operation without control produced the fastest movement, officer control produced the next fastest, and the traffic-actuated resulted in the slowest movement of the three. All three, however, produced a faster movement than any of the fixed-time controls with the exception of the 15–15 setting.

No-control offers the highest degree of flexibility of movement possible, since vehicles from a number o entering ways may be in the intersection at one time. There is, of course, a limit to the volume which an uncontrolled intersection may handle, but this limit was above the range of these observations. During the study rates of over 1,000 vehicles per hour for 15-minute periods in one direction on Constitution Avenue and of over 600 per hour on Seventeenth Street in one direction were recorded. On a per lane basis this would be over 300 per hour per lane for each direction.

The maximum rate for the entire intersection recorded for any one 15-minute period was 2,260 vehicles, divided as follows: Constitution Avenue west-bound 740, east-bound 710; Seventeenth Street north-bound 430, south-bound 380. On a per lane basis these figures become (approximately) 250, 240, 220, and 190 vehicles per hour. Assuming that these figures per lane can be applied to a simple 2-lane intersection, the volume would be then 900 vehicles per hour, with 55 percent on the heavier street. For a 4-lane intersection, these average figures per lane represent an hourly rate of 1,800 vehicles per hour. Such a comparison must be considered as merely an indication, however, since the operating characteristics of a 2-lane or of a 4-lane intersection might be quite different from those of this 6-lane and 4-lane intersection.

During the studies no accident or even near-accident was noted. Toward the late afternoon, when the traffic began increasing, free movement became more difficult and some confusion was apparent. During this period of heavier traffic the vehicle time did not show any marked increase, but it was obvious that a further increase in traffic would increase the confusion to the point of congestion. The vehicle time for the no-control method was considerably less than that for officer control, the next fastest, which appears to be due to the small number of stopped cars. The shortest light control, and the most efficient officer control must stop some vehicles while under no-control almost all vehicles keep moving. Vehicle movement under the traffic conditions studied here, even with the necessary slowing down, is bound to be faster than any sort of regimented movement.

AUTOMATIC TRAFFIC LIGHT ADAPTS CONTROL TO CHANGING CONDITIONS

The officer assigned to direct traffic during this study was exceptionally able and active, and while no doubt his efficiency was increased by his knowledge that the vehicles under his direction were being timed, he would unquestionably direct traffic in a highly efficient manner regardless of scrutiny. He used a semaphore, common to Washington, for indicating the movements, and compelled observance of his signals by vigorous use of his whistle. The longest tour of duty was approximately 2 hours, but he believed that with opportunity for a short rest period at about 2-hour intervals he could maintain this same efficiency throughout an entire day. These studies may not be typical of what an average officer does, but they show what an efficient officer can do. The time per vehicle was 3.6 seconds more than that under no-control, but was 1.6 seconds per vehicle less than the fastest light-control method.

The traffic-actuated control is listed fourth in order of time per vehicle, following the 15–15 setting of the fixed time, but the times of the two are practically identical, 31.5 seconds for the fixed time control and 31.6 seconds for vehicle-actuated light control. The 15–15 setting was entirely satisfactory for the volume of traffic present. It is doubtful if this short cycle would be effective in very heavy traffic, due to the tendency of vehicles to run through the amber and into the red interval under such conditions. Such action was noticeable for the extremely short cycle lengths, even with the moderately heavy traffic encountered during these observations. With an impatient rush-hour traffic such disregard of the signal would result in dangerous confusion.

It is emphasized again that it is not intended to prove by this study that any one type of control is superior to all others under a variety of conditions. The purpose is to find what specific conclusion can be reached from a study under fixed conditions except for cycle arrangement. The fact that one fixed-time cycle shows a time per vehicle of slightly less than that for the traffic-actuated control under the same condi-

tions should not be considered so significant as the fact that the shortest fixed-time cycle shows the least delay of all the fixed-time cycles. The most important feature of traffic-actuated control is its extreme flexibility, and the automatic adaptation to changing conditions. It is significant that this flexible control automatically adjusted itself to practically the same efficiency as that of the fixed-time control best adapted to the conditions existing during the hours studied at this particular location. In addition, it was always ready to control traffic of a different pattern with equal efficiency. No intersection operates under the same traffic conditions from day to day, and but few from hour to hour.

OPERATION OF AUTOMATIC CONTROL DESCRIBED

A brief description of the operation of this traffic-actuated control will aid in understanding the significance of the figures in following analyses. The signals themselves are the same as in any other light control, but instead of being actuated by a cam or other constant-speed mechanism, the changes are actuated by the vehicles passing through the intersection. The interval lengths are determined by the vehicles approaching the intersection, which results in continually varying cycle lengths. The particular device used is ingenious in conception and simple in operation, the various phases of the signal being initiated by the functioning of electric condensers in the control mechanism.

Assume that traffic is flowing on street A, and no traffic is waiting on street B. This traffic will continue to flow without interruption until the arrival of a car on street B requires the right-of-way for that street. On crossing a detector placed at some distance from the intersection (see fig. 1), the car on street B imparts an impulse to the control mechanism, after which street B receives the green signal by virtue of one of two reasons. First, a gap may occur in the traffic on street A. If this gap in traffic is of such length that no vehicle crosses the detector on street A for a period equal to the "vehicle interval", the length of which may be varied in the control mechanism, the right-of-way is given to street B immediately. If no such gap occurs in the traffic on street A, the right-of-way will revert to street B after a certain time called the "maximum interval", which also may be varied in the control mechanism.

One other feature of importance is the "initial interval." This period is to allow stopped vehicles waiting for the signal change a greater time to clear the intersection than is necessary for vehicles crossing the detector at a normal driving speed.

The efficiency of this equipment depends to a considerable degree on the setting of the various interval lengths. During this study the settings were as follows: Vehicle interval on each street, 5 seconds (the lowest available); initial interval on each street, 7 seconds; maximum interval on Constitution Avenue, 30 seconds, which allowed a maximum time of 30 plus the initial interval, or 37 seconds; and the maximum interval on Seventeenth Street, 35 seconds, or a maximum time of 42 seconds. The longer maximum on Seventeenth Street was necessitated by the rush of traffic on this street for a short period just before 4:30 p.m., before traffic on Constitution Avenue had shown a marked increase. These settings were those used by the Park Police, and are believed to be the most satisfactory for this location. Typical conditions under traffic-actuated control are illustrated in figure 7.

Officer control and traffic-actuated control are much alike in their operating characteristics. Analysis of these two types offers interesting comparisons, and throws more light on the result of increasing the interval and cycle lengths. Table 3 summarizes certain features of the flexible control methods.

TABLE 3.—*Comparison of flexible control methods*

Control	Average cycle				Average vehicle time	Vehicle time for same cycle with fixed light control	Benefit of flexibility	
	Constitution Avenue	Seventeenth Street	Ratio	Total			Time	Percent
	Seconds	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	*Seconds*	
Officer	26.8	18.0	1.5–1	44.8	29.9	32.5	2.6	8.0
Vehicle-actuated light	23.4	17.1	1.4–1	40.5	31.6	32.1	0.5	1.5

OFFICER CONTROL AND TRAFFIC-ACTUATED CONTROL COMPARED

The cycle lengths of table 3 are the averages for all studies on these control methods, and the vehicle times for fixed-time cycles of the same interval were computed with equation 1. Two factors are important in this table. First, each flexible control method shows a slight advantage over what might be expected from fixed-time control of the same time intervals. This does not mean that flexible control results in faster movement than fixed control of some other interval length (see results of 15–15 timing in table 2), but it shows the advantage of flexible control over a comparable fixed-time setting. Again it must be remembered that the

FIGURE 7.—(UPPER) CONDITION UNDER WHICH TRAFFIC ACTUATED CONTROL IS EFFECTIVE AND (LOWER) TYPE OF TRAFFIC WHICH WILL HOLD THE GREEN LIGHT UNLESS "VEHICLE INTERVAL" IS SHORT AND RESULT IN INCREASED AVERAGE VEHICLE TIME.

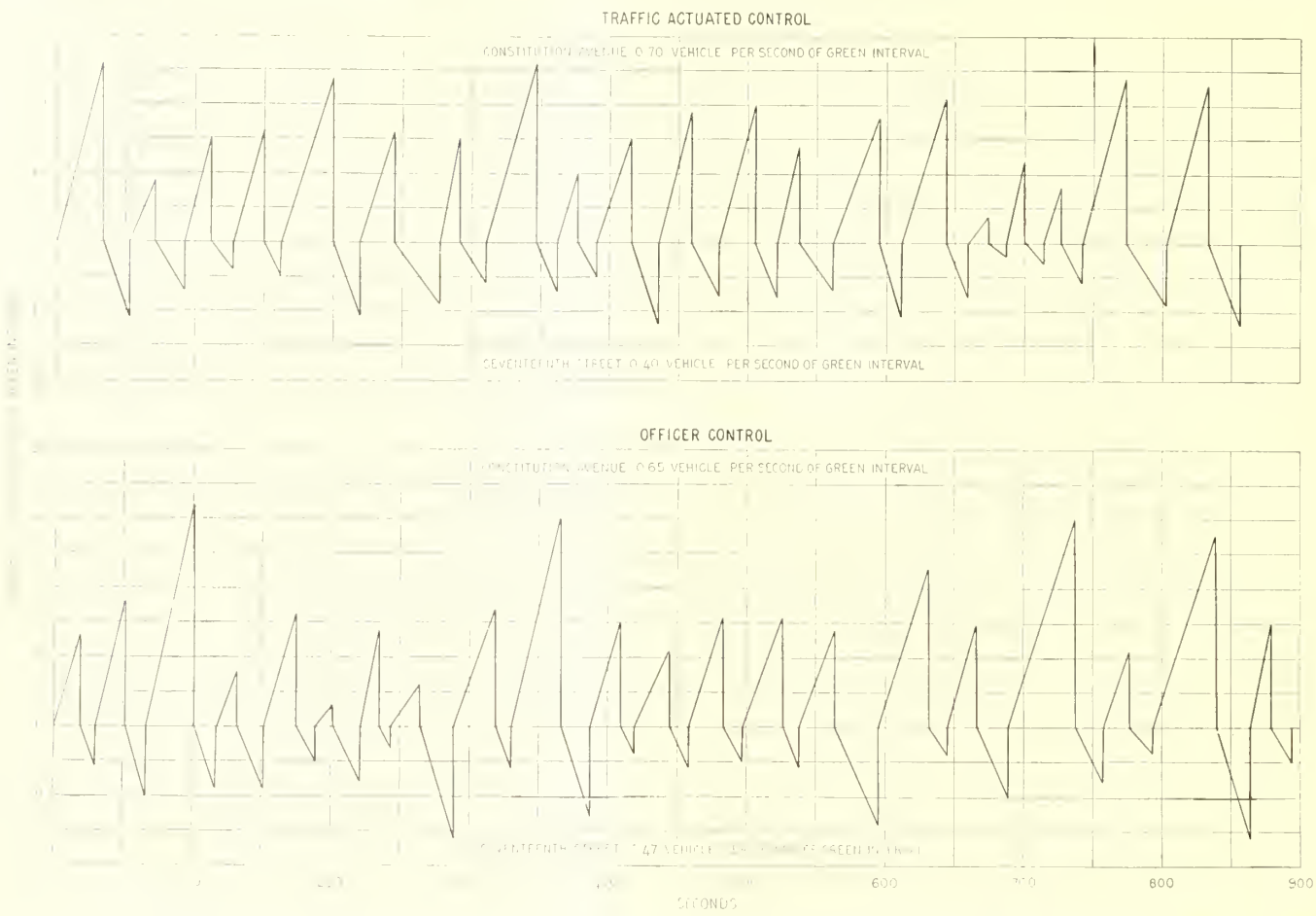

TRAFFIC ACTUATED CONTROL
CONSTITUTION AVENUE 0.70 VEHICLE PER SECOND OF GREEN INTERVAL
SEVENTEENTH STREET 0.40 VEHICLE PER SECOND OF GREEN INTERVAL
OFFICER CONTROL
CONSTITUTION AVENUE 0.65 VEHICLE PER SECOND OF GREEN INTERVAL
SEVENTEENTH STREET .47 VEHICLE PER SECOND OF GREEN INTERVAL
600
700
800
900
SECONDS

FIGURE 8.—MOVEMENT OF TRAFFIC BY CYCLES, DISCHARGE ON CONSTITUTION AVENUE, PLOTTED AS POSITIVE VALUES AND THAT ON SEVENTEENTH STREET AS NEGATIVE VALUES. SLOPE OF LINES INDICATES DISCHARGE RATE.

greatest advantage of flexible control is the ability to meet varying conditions; it is commendable in that it approaches the best fixed-time control for these particular conditions.

The second and perhaps more significant fact is that officer control produces faster movement than traffic-actuated control, even though a longer cycle is used. This is contrary to what would be expected from the analysis of the fixed-time studies and is worthy of further analysis. Table 3 shows that the officer divided the time between Constitution Avenue and Seventeenth Street in the ratio of 1.5 to 1, whereas the traffic-actuated control gave Constitution Avenue only 1.4 times as much time as Seventeenth Street received. With cycle lengths as near alike as these, the one which allowed the light-traffic street the smaller proportion of the cycle should show the lowest weighted average time per vehicle. Further information with respect to the details of these methods may be obtained from figure 8, which shows the time interval variation during random 15-minute periods for the flexible control methods and the corresponding discharge rates.

In preparing figure 8 periods were chosen in which the average time interval closely approximated the average time interval for the entire study of each method. The slope of the lines shows that the time interval generally varied with the number of vehicles. It is apparent that where the interval length on Seventeenth Street increases, the number of vehicles appears to follow this increase more closely in the case of the officer control than for the traffic-actuated control. Accordingly, for the periods selected here, further analysis was made of the long cycles. In each case the length of interval and the number of vehicles were determined for all intervals of over 30 seconds on Constitution Avenue, and of over 20 seconds on Seventeenth Street. The results of this analysis appear in table 4.

TABLE 4.—*Comparison of officer control and traffic-actuated control for extended intervals*

	Officer control	Traffic-actuated control
Constitution Avenue:		
Average vehicles per second	0.65	0.70
Vehicles per second in long intervals (over 30 seconds)	.69	.67
Difference in vehicles per second between average conditions and long intervals	+.04	−.03
Above difference as a percentage	+6.2	−4.3
Seventeenth Street:		
Average vehicles per second	.47	.40
Vehicles per second in long intervals (over 20 seconds)	.52	.38
Difference in vehicles per second between average conditions and long intervals	+.05	−.02
Above difference as a percentage	+10.6	−5.0

Table 4 shows that the officer control not only was relatively more efficient in the long cycles than was the traffic-actuated control but was actually more efficient in the longer cycles than in the shorter ones. This indicates that the vehicle-actuated control held the right of way too long on one street for scattering traffic, whereas the officer did not give a long cycle unless the traffic demanded it. This is a further reason for the apparent inconsistency between officer and traffic-actuated control, where the method showing the longer

average cycle showed the faster movement. From this two conclusions may be deduced. (1) The traffic-actuated control did not have quite the flexibility of officer-control, and (2) the officer might have been more efficient had he been able to maintain a shorter average cycle. The fact that the use of the longer cycles was more efficient than that of the shorter ones indicates usage of over average intervals only when traffic demanded it, and that short and average cycles could have been still shorter.

SHORT "VEHICLE INTERVAL" DESIRABLE IN AUTOMATIC CONTROL

Since the operation of the traffic-actuated control is dependent entirely on the spacing of traffic, it is of interest to examine the flow of traffic on the two streets. Constitution Avenue, during the hours studied, carries approximately 1,390 vehicles per hour, and Seventeenth Street carries about 670. This means that at evenly spaced intervals vehicles would arrive on Constitution Avenue about every 2.6 seconds, and on Seventeenth Street every 5.4 seconds. Of course vehicles do not arrive so regularly, and it is because of the grouping tendency that traffic-actuated control is effective. However, on a city street speeds are more nearly uniform than on an open highway, and with closely spaced intersections, traffic may enter or leave a street at frequent intervals. For these reasons, with the relatively heavy volume recorded at this intersection, the flow was more nearly uniform, or continuous, than grouped.

On Constitution Avenue, for example, vehicle-actuated control, operating with a vehicle interval of 5 seconds, permits the clearing out of the stopped cars, and then may hold the right of way while the traffic on the street flows at only its normal frequency. This is not so serious on the heavily traveled street as on the one with lighter traffic, as was brought out in figures 4 and 5, but it does work to the disadvantage of this type of control.

The two detectors on a street are connected to the dispatcher in parallel with one another. An impulse on one, followed by an impulse on the other 4.9 seconds later, followed by another impulse on the first detector at another 4.9-second interval could hold the right of way with a spacing of vehicles in both directions of practically 10 seconds. While this extreme case is seldom found, it was frequently noted that the right of way was held for only a scattering of cars. It has been shown that it is wasteful of time to permit the traffic on even the more important street to flow any considerable length of time after the waiting vehicles have cleared the intersection. A control, to be the most effective, should not cause waiting for a gap in the normal flow of traffic on one street to permit the cross traffic to flow, but should revert to the other street as soon as the traffic on the first street becomes normal provided there is not too great a disparity in traffic on the two streets.

A control mechanism of the type used should be arranged so as to be set for a vehicle interval as low as 3 or even 2 seconds for traffic such as was observed. Five seconds was the shortest time available for the vehicle interval in this particular instrument, and no observations with shorter intervals could be made. It is reported, however, that a reduction of from 8 to 5 seconds in one particular case (Providence, R.I.) resulted in a material improvement in the flow of traffic, and it is believed that reduction of from 5 to 3 seconds, would result in reduced delay at this intersection.

VEHICLE DELAY DETERMINED

In the foregoing discussion of efficiency it may have been noted that comparisons were always made of the vehicle time. This was what was actually measured, and this offered the fairest comparison of efficiency. However, what is more important to certain interests is the delay to traffic caused by various types of control. Having found the total elapsed time for the average vehicle to traverse the 600 feet included in this intersection zone, to find the delay it is necessary only to subtract from the elapsed time the time required to traverse the distance at normal driving speed. In the District of Columbia the legal speed limit is 22 miles per hour, and at this speed a vehicle requires 18.5 seconds to travel 600 feet. Assuming 22 miles per hour as the normal driving speed on the streets approaching this intersection, the delay under various conditions is as shown in table 5.

TABLE 5. *Delay under various control methods based on assumption of 22 miles per hour as normal driving speed*

Control method	Vehicle time	Vehicle time corresponding to normal speed	Delay	Vehicles per hour	Delay per hour (car minutes)
	Seconds	*Seconds*	*Seconds*		
No control	26.3	18.5	7.8	2,000	260
Best method of control recorded	29.9	18.5	11.4	2,000	380
60–60	45.4	18.5	26.9	2,000	900

The figures in table 5 are for normal daytime traffic only. Studies made during certain rush hours, indicate that the time of an average vehicle in rush hours could not be less than 50 seconds, under efficient control. Using this somewhat arbitrary, but conservative figure, the delay per vehicle during the rush hour, assuming the same driving speeds as during the lighter midday traffic, must amount to a minimum of 30 seconds per vehicle, and for the 3,500 vehicles recorded from 4:15 to 5:15 p.m., causes a delay of 1,750 car minutes per rush hour. Under the best control methods at this particular intersection, for a 10-hour day, including two rush hours, the minimum delay which can be expected is as follows:

8 normal hours, 8×380 = 3,040 car-minutes
2 rush hours $2 \times 1{,}750$ = 3,500 car-minutes

6,540 car-minutes

It is not practicable to fix a definite value for the economic loss incurred by traffic delays. Various agencies have advanced theories applying to particular cases which no doubt have definite merit, but which cannot be applied generally. Costs have been computed from such basic assumptions as the amount of gasoline consumed while waiting in traffic, and the increased wear on vehicles in acceleration and deceleration, and also from such arbitrary assumptions as the value of the time lost by the occupants of even purely pleasure vehicles. These range from extremely low figures to values as high as 5 cents per car-minute, obviously fantastic for the average case. For these reasons no value of time lost is assumed to be applicable to general conditions, but it is of interest to investigate how much this loss might amount to under various assumed values of the cost of delay. Figures for time lost in a 10-hour

day (8 a.m. to 6 p.m.) appear in table 6. It will be noted that these figures are based on the most efficient control. If a similar intersection were operated under poor control, the delay would be much greater.

An interesting feature with respect to delay is its distribution through the day. Using the 10-hour day previously referred to, it may be seen that of the 6,540 car-minutes delay, 3,500 car-minutes, or over half the delay, was incurred during the two rush hours. Any reduction in delay during these periods will have a marked effect on the time loss for the whole day.

TABLE 6.—*Economic loss incurred in a 10-hour day based on assumed values per car-minute*

Assumed cost per car-minute	Delay per day (car-minute)	Per day	Per year (300 days)
Cents			
½	6,500	$32.50	$9,750
1	6,500	65.00	19,500
1½	6,500	97.50	29,250
2	6,500	130.00	39,000

SUMMARY OF CONCLUSIONS

(1) For the traffic volume observed in this study (about 2,000 vehicles per hour) operation of the intersection without control incurred the least delay to traffic. Of all the control methods, officer control permitted the fastest movement of traffic, closely followed by the shortest fixed-time control, and traffic-actuated control.

(2) Under fixed-time control, a very marked increase in delay resulted from lengthening the cycle. For certain of the short cycles with unequal intervals on the two streets, reversing the proportioning of the cycle had less effect on the time of the average vehicle than did retaining the same proportioning but doubling the cycle length.

(3) The flexible control methods showed efficiency equal to or better than the most efficient fixed-time control, but it is believed that both the officer control and traffic-actuated control could be more efficient than indicated here.

(4) During the course of a 10-hour day from 8 a.m. to 6 p.m. as much delay is incurred during 2 hours of peak traffic as during the remaining 8 hours.

APPENDIX.—METHODS OF SURVEY AND ANALYSIS

It is believed that a description of the methods of collecting and analyzing the information presented in this report will be of interest, since as far as is known, few, if any such detailed analyses of vehicular movement have been attempted. As previously noted, delay to traffic at an intersection includes time lost in waiting, decelerating and accelerating, or in driving at a low speed in anticipation of a signal change. While for comparative purposes the determination of but one of these factors may be sufficient, if it is desired to determine the absolute time loss all the factors must be considered. The distance within which delay occurs is defined as the zone of influence of the intersection, and varies with local conditions. The difference in the average time required to traverse this zone of influence and the time required to traverse the same distance at a normal driving speed represents the delay caused by the intersection.

ARRANGEMENT OF OBSERVERS AND APPARATUS DESCRIBED

Since the actual timing of a particular vehicle through the intersection would be difficult, and because, were this feasible, only a small sample of traffic could be timed a method was developed using a graphic time recorder which permitted the timing of all vehicles using the intersection. The instrument (fig. 9) consisted of a strip chart operated by clockwork at a known speed, under a number of pens, electrically operated by the closing of various circuits. Any pen could be actuated by a telegraph key at any point to which it was convenient to extend wires. It was possible to station an observer at a predetermined point just outside the zone of influence who recorded each vehicle approaching the intersection as it passed his post. Other observers, each equipped with a key in series with a different pen, then recorded these same vehicles as they left the intersection zone, one observer being stationed on each of the three streets by which the vehicle might leave the intersection.

In this study the observers (fig. 10) were located 300 feet from the intersection. An "incoming" observer on each street recorded all vehicles entering the intersection zone as they passed his post, and accounted by special notation for all abnormal movements, such as parking, between his post and the intersection. To record the outgoing traffic, each "outgoing" observer, one for each street, was equipped with three keys, one for each of the three streets from which the outgoing vehicles might have entered the intersection. It was

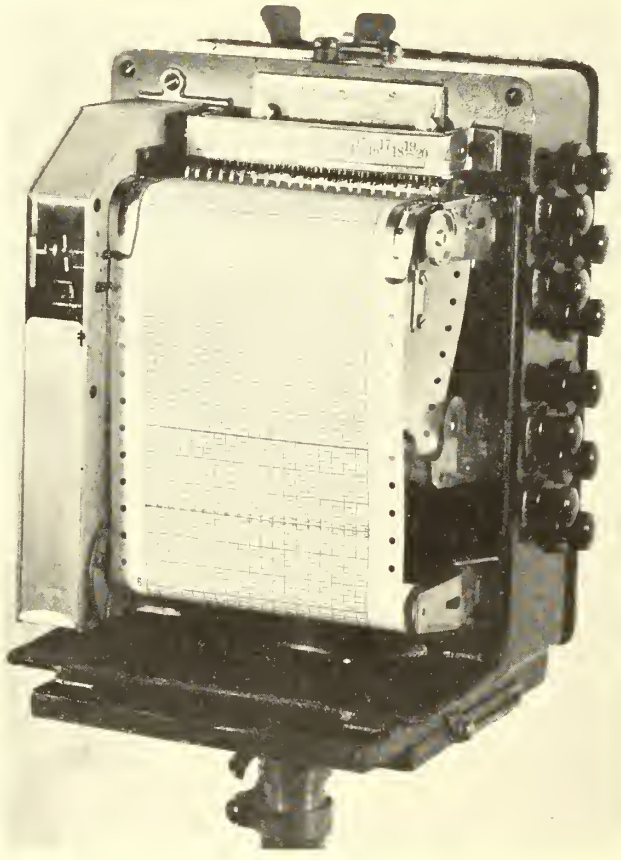


FIGURE 9.—THE TIME-RECORDING MACHINE.

FIGURE 10.—RECORDERS OF INCOMING AND OUTGOING TRAFFIC AND TRUCK CARRYING RECORDING APPARATUS.

necessary for this observer to classify the vehicles as they left the intersection, according to whether they came straight through or from the right or left, and record them with the proper key when they passed his post. This recording was simplified somewhat by the traffic control, for except for unusually fast or unusually slow vehicles near the beginning or end of the green interval on either street, the vehicles during one interval were all "through" and during the other interval were all from either "left" or "right", and it was seldom that the three classifications need be noted simultaneously.

Two observers and 4 keys at each post required 8 observers and 16 keys for a complete record of the operation of all traffic using the intersection. The time recorder was equipped with 20 pens, which permitted the use of four for other purposes. A relay in the light-control box (fig. 11) actuated a pen and automatically recorded the cycle of the signal light. Two pens were used to record all vehicles as they passed the crosswalks on entering the intersection, and the remaining pen was kept ready for emergency, to provide for the occasional failure of one of the pens to function properly.

ANALYSIS OF RECORD CHARTS PRODUCES ACCURATE DATA ON VEHICLE TIME

The time required for the average vehicle to pass through the intersection was determined as follows: Assume vehicles approaching the intersection on a given street are recorded as they enter the zone of influence at a, b, c, d, and e seconds after any selected

FIGURE 11.—TRAFFIC CONTROL BOX.

zero point on the chart. These vehicles are recorded on the various "outgoing" pens at times a′, b′, c′, d′, and e′ seconds after this same zero point. The time for each of the vehicles to traverse the intersection is $(a'-a)$, $(b'-b)$, $(c'-c)$, $(d'-d)$, and $(e'-e)$, and the total time for all vehicles is the sum of these expressions, or $(a'+b'+c'+d'+e')-(a+b+c+d+e)$. Dividing by the number of vehicles gives the average time per vehicle.

Whether one vehicle of this group passes another is immaterial. Assume the first vehicle passed the second and third. The times of the individual vehicles would then become $(a'-b)$, $(b'-c)$, $(c'-a)$, $(d'-d)$, and $(e'-e)$ and as before, the total time would be $(a'+b'+c'+d'+e')-(b+c+a+d+e)$. In using this method it is essential that the records on the "outgoing" and "incoming" pens, respectively cover the same group of vehicles. For each period of study the "incoming" and "outgoing" records must begin with the same vehicle and end with the same vehicle. Every vehicle recorded as "incoming" must be recorded as "outgoing" at some point, in order that the total "outgoing" vehicles equal the "incoming" total. Having ascertained that the "incoming" and "outgoing" records start with the same vehicle, and also stop with the same vehicle, the movement of vehicles within this group relative to one another is immaterial, and it is only necessary to account for all vehicles entering and leaving, and to discount by special notation all unusual movements such as parking, stopping for other than traffic delays, entering or leaving the zone of influence at a point other than that at which the observers are stationed.

Various methods may be used to indicate the vehicle with which the record should begin or end. At this intersection each of the four directions was considered in order and the observers were signaled from the intersection as to the proper vehicle, usually the first after a gap in the traffic, with which to start recording. Often the intersection was entirely clear of traffic entering from a particular direction, and this could be noted on the chart by the recorder by special manipulation of the key. Such a "break" is helpful in analysis of the chart, for if no vehicles are in the zone of influence at any given time, the succeeding records must necessarily begin with the proper vehicle.

For ease in analysis the circuits were so arranged that for the "incoming" record appearing on pen 1, the "outgoing" records for the through, right and left movements appeared on pens 2, 3, and 4, respectively. Similar groups of four pens accounted for the movement on the other streets. The record on each chart covered from 2 to 3 hours. To facilitate analysis and to investigate the short time fluctuation of traffic, the record was broken up into periods approximately 15 minutes in length. Experience showed that shorter periods often gave disproportionate results, and much longer periods made analysis of the charts more difficult.

Charts were examined carefully and all special notations made by the recorders during the study properly accounted for. The "outgoing" record on each street was checked against the "incoming" record, which made mistakes immediately apparent. Errors in recording heavy traffic are bound to occur, but due to the character and training of the observers mistakes were not frequent. The largest portion of mistakes could be traced to the "outgoing" recorder using the wrong key, since vehicles were classified at the intersection but were not recorded until they passed the observers post. For example, should a left turn be recorded as a right turn, the "outgoing" record from one street would show a surplus of one car, which would correspond to a deficit in the "outgoing" record from another street. Such errors could be recognized and a correction made on the record chart. Other errors could be isolated within a short period by means of the recorders "breaks," and correction made. With the "incoming" and "outgoing" records in agreement, the average time per vehicle was computed using the method outlined above.

SIMPLER METHODS REQUIRED FOR GENERAL USE

The purpose of this study was to determine the effect of the variation in control methods under conditions of traffic as nearly constant as possible. After computation of the average running time for each 15-minute period, this time was plotted against the volume for the particular period for each control method, as a safeguard against including times influenced by any external factors. While certain periods showed rates of flow deviating somewhat from the average, there were few in which there was possibility of tracing any change in running time to the variation in volume, and these were not considered in the final analysis.

Investigation was made of the effect of left turns. It was found that left turns which, on Constitution Avenue, the heavier traveled street, ranged from 5.6 percent to 18.0 percent of the total traffic, and on Seventeenth Street ranged from 20.0 percent to 42.8 percent, had no apparent effect on the average running time, within the range of volumes observed. While no doubt left turns in congested traffic will have a marked effect on traffic delays, no such effect was noted in this moderately heavy traffic. Pedestrian traffic was negligible during the hours included in these observations.

The methods used permitted the accurate timing and detailed analysis of the speed of traffic. Probably no more accurate or inclusive information could be obtained with methods now available to traffic investigators. The question arises as to whether such a detailed analysis is necessary. Obviously a method similar to this is essential if all the delay to traffic is to be accurately analyzed, but there may be some question as to the necessity of going to such extremes in accuracy and detail to obtain satisfactory information at an ordinary intersection. Such a study requires, if the traffic is heavy, a minimum of 8 recorders, a supervisor and a time-recorder operator. Lighter traffic may permit the reduction of the recorders to 6, or possibly to 4, or for certain types of studies the intersection might be analyzed in one direction at a time, but whatever the arrangement of personnel, there is required a laborious and somewhat complicated laying out of wires and assembling of equipment. For a study as detailed as the one made a light truck was essential, and this together with the time recorder and its accessories represents a considerable investment. More than average accuracy and experience on the part of the recorders is required.

Obviously, this method of analysis is not adapted to widespread use. It is hoped that these studies will provide material which will point to simpler and more generally applicable methods of analysis which will at least provide information sufficiently accurate for comparative purposes. It may be possible to determine factors, which can be applied to less extensive data than were collected and thus provide satisfactory figures on delay.

AN IMPROVED METHOD OF MEASURING SPEED OF TRAFFIC

Reported by E. H. HOLMES and LAWRENCE S. TUTTLE, Assistant Highway Economists, United States Bureau of Public Roads

Subsequent to the studies described in the preceding papers in this issue of PUBLIC ROADS, there has been a further development of field methods used in the determination of vehicle time loss. The method used in the New Jersey Viaduct study consisted of recording the licenses and the times of passage of vehicles at both ends of the route under consideration. Since the length of the route was about 4 miles, recording of time to the nearest minute was sufficiently precise. A large proportion of the vehicles were recorded in this way and the average travel time was determined from the travel time of the individual vehicles.

In the method used in the Constitution Avenue and Seventeenth Street study the travel time per vehicle was not determined by averaging the time for individual vehicles. Instead the average time was obtained by dividing the aggregate time for all vehicles by the number of vehicles. Here the distance was but a few hundred feet rather than several miles, and so the time of passage of the vehicles had to be noted precisely. The required precision was obtained with the electric time recorder, which permitted timing to the nearest second. While this type of study was very precise, its accuracy was dependent on the "incoming" and "outgoing" records being in exact agreement.

The two methods, while entirely satisfactory in each special case, are limited in their application. The first method can be used only where the distance over which the vehicles are to be timed is great enough that recording to the nearest minute, or possibly to the nearest half minute, will give the elapsed time with sufficient precision. This method is advisable only when most of the traffic passes both observation points, for otherwise the sample is small. Such diversions or entrances of traffic may be accounted for, however, by the use of additional observers at the diversion points. The second method is useful only over a short distance where both ends of the section of street or highway are readily visible from a selected point, which sets a practical limit of about 1,000 feet to the length of the section. Difficulty is encountered if there is a considerable amount of stopping or parking within the section studied, or if vehicles enter or leave at points other than those at which the observers are located. Stores, filling stations, or building entrances make such a study very difficult, while another street intersection within the "zone of influence" of the intersection being studied excludes this type of study entirely.

Observations at intersections in business sections recently investigated were so complicated by business stops and by the long lines of vehicles awaiting signal changes during rush-hour traffic that neither method was suitable. However, a combination of the two methods proved successful.

Under this system, the licenses are recorded as the vehicles enter and leave the "zone of influence" of the intersection, and the time of passage of each vehicle is noted on the time-recorder chart by the closing of a circuit with a telegraph key as in the regular intersection study. This type of recording requires two observers at each post for each direction. One reads aloud the license number and simultaneously records the time of passage of the vehicle with the telegraph key. The other writes on a ruled sheet the number read by the first observer.

In order to correlate the time record appearing on the strip-chart with the license record shown on sheets, it is necessary for the observers to designate at frequent intervals a certain car as a "check" vehicle. The license number of this vehicle is checked or otherwise noted on the sheet, and its time of passage indicated on the time-record chart by a special manipulation of the telegraph key. Usually it was found convenient to check every tenth vehicle in this way, with distinctive notations for the fiftieth and hundredth vehicles. To facilitate this correlation of the records, the sheets used by the observers were previously ruled, and the tens, fifties, and hundreds indicated on them. Recording of these "check" vehicles was not difficult, since the recording observer, after recording nine numbers, would announce to the "reading" observer whose eyes were on the traffic, that the following vehicle would be the tenth.

If the distance over which the record is being made is short, as at an intersection, the last 3 digits of licenses will be sufficient for identification of vehicles. Using this 3-digit recording, 2 observers can record as many as 800 vehicles per hour with little difficulty.

Analysis of the information obtained requires first matching the numbers shown on the "incoming" sheets with those shown on the "outgoing" sheets and eliminating all those which do not appear on both sheets within a reasonable elapsed time. Then, since the time record on the chart is correlated with the license record on the sheets, these same vehicles may be eliminated from the chart. With the vehicles not matching removed from the record, the aggregate time for all vehicles is obtained by the summation of the "incoming" and "outgoing" times, as described in the preceding paper, and the average trip time found from this figure.

In addition to providing an accurate record of "incoming" and "outgoing" times from which an aggregate time for all vehicles may be found, this type of study, because of the possibility of identification of particular vehicles, permits a much more detailed analysis than is possible with the regular intersection analysis. For instance, it is very easy to time separately right and left turns, or to make a complete analysis of the movement of vehicles through multiple intersections.

Use of this method is not confined to intersections. It is ideally adapted to analysis of traffic on open highways. It is possible to determine the average speed maintained over a given section of highway and how the speeds of the individual vehicles vary from the average speed. The effect of changes in traffic volume on average speed and freedom of movement can be studied. Determinations may also be made of the capacity of traffic lanes under ideal conditions, and how their capacity may be affected by grades, curves, and road widths.

To conduct such a study on the highway it is necessary only to station observers at the two or more critical points between which information is desired. The distances which may be included are dependent only on the length of wire which it is practicable to lay out for the study.

By gradual development and combination of proven methods, it is now possible to analyze accurately the movement of traffic over distances up to several miles. Practically every condition of traffic flow may be studied by some combination of the methods described.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS I—PROJECTS ON THE FEDERAL-AID HIGHWAY SYSTEM OUTSIDE OF MUNICIPALITIES

AS OF JANUARY 31,1934

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS I PROJECTS ON THE FEDERAL AID HIGHWAY SYSTEM	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS I PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	4.155.067					4.841.734.76	2.451.108.12	1.990.626.64	26.3	250.9	592.998.62	58.2	1.140.960.26
Arizona	3.804.731	305.538.61	305.538.61		21.6	3.049.361.87	2.667.079.99		26.9	228.0	182.322.13	3.3	849.790.27
Arkansas	3.374.167	18.339.52	9.849.52	8.490.00	.6	1.398.998.25	1.020.035.88	378.962.37	35.3	56.9	592.933.47	42.6	1.751.348.13
California	7.803.677	92.145.42	74.403.20		1.8	7.562.941.95	5.894.074.97		24.3	227.7	1.131.999.51	43.9	703.199.32
Colorado	3.437.265	500.887.00	500.887.00		33.3	1.482.693.51	1.472.113.16		34.7	68.5	295.795.28	12.0	1.168.469.56
Connecticut	1.404.213					1.230.901.72	1.055.122.78	175.778.94	5.6	24.4	309.548.49	6.3	39.241.73
Delaware	909.544	133.532.30	133.532.30		15.4	742.815.70	742.815.70		21.3	17.4			33.196.00
Florida	2.615.917					2.572.665.70	1.791.781.49	750.274.11	33.5	92.6	555.647.43	18.3	268.488.08
Georgia	5.045.592					1.759.445.32	1.759.445.32		33.6	96.1	1.123.450.05	77.5	2.162.696.63
Idaho	2.243.125	274.499.49	273.270.57		35.5	1.358.904.11	1.345.399.96		43.6	95.1	146.518.90	13.4	477.935.57
Illinois	4.431.348					922.816.03	922.816.03		10.2	21.8	563.855.05	8.2	2.944.676.92
Indiana	4.717.786					1.974.416.80	1.974.416.80		28.7	67.5	1.124.202.64	32.2	1.619.166.56
Iowa	5.027.830	707.037.81	704.400.00		30.0	3.948.094.43	3.719.700.00		30.2	209.8	492.480.00	22.8	111.250.00
Kansas	5.044.802	149.657.30	149.657.30		49.5	3.099.840.41	3.095.879.28		32.6	285.4	1.799.265.42	66.4	
Kentucky	3.608.332	90.579.10	90.579.10		2.9	1.823.231.23	1.823.231.23		33.0	142.1	1.155.509.82	87.9	539.011.85
Louisiana	2.914.295	12.626.07	12.626.07		.4	2.026.345.04	1.519.694.04		22.2	37.0	784.551.81	30.5	597.423.08
Maine	1.684.959					1.161.321.30	1.125.754.35		36.5	35.5	218.070.16	7.4	341.134.49
Maryland	1.782.263					801.143.78	791.495.03		7.1	14.8			990.767.97
Massachusetts	1.101.716	153.936.88	83.952.64	69.984.24	4.2	1.112.362.46	796.702.82	315.659.64	17.8	32.8	112.698.19	.8	108.362.35
Michigan	5.094.491	35.500.00	35.500.00		1.2	2.449.550.00	2.449.550.00		33.5	130.6	2.276.866.00	88.0	332.575.00
Minnesota	5.115.153	1.232.279.59	1.232.279.59		257.2	1.819.309.81	1.819.309.79		30.2	266.3	515.258.72	141.2	1.548.304.90
Mississippi	3.489.337					2.909.936.11	1.564.495.62	1.306.808.55	10.7	145.0	460.775.41	56.7	1.484.065.97
Missouri	6.090.153	318.173.69	219.506.32		20.1	3.319.647.58	3.004.621.74	25.499.64	41.9	105.2	1.546.982.18	56.3	1.319.042.76
Montana	4.463.849	218.002.08	218.002.08		34.0	3.708.805.40	3.435.926.52	192.878.88	31.8	268.5	656.603.46	40.7	153.316.94
Nebraska	3.914.481	381.123.97	255.420.54		144.8	3.799.643.25	2.945.410.14	108.538.00	39.2	176.3	557.688.11	57.3	155.962.21
Nevada	2.909.387	272.235.60	272.235.60		60.3	1.466.705.48	1.466.705.48		22.4	131.4			1.170.445.92
New Hampshire	725.739					545.335.31	539.650.31		28.5	9.2	99.120.09	1.6	86.968.60
New Jersey	3.065.137					1.098.555.56	1.098.555.56		9.3	19.3	845.917.99	10.3	1.120.663.45
New Mexico	2.896.487	1.072.779.31	1.072.779.31		139.0	1.311.063.23	1.311.063.23		2.5	94.3	426.264.11	44.4	86.360.35
New York	10.830.099	117,100.00	117.100.00		3.8	9.197.850.00	8.225.175.00	315.000.00	24.2	201.8	1.858.230.00	29.6	629.594.00
North Carolina	4.761.147	444.700.10	222.350.08	222.350.02	35.1	1.289.338.80	959.681.10	329.657.70	33.7	149.1	1.578.487.71	218.7	2.000.628.11
North Dakota	2.902.224	500.761.09	500.757.58		249.7	861.077.82	861.077.82		10.9	230.6	1.165.294.81	355.6	375.093.79
Ohio	6.968.066	213.370.00	213.370.00		4.8	5.686.265.00	5.568.315.00		34.3	147.5	883.975.00	21.4	302.406.00
Oklahoma	4.608.399	122.702.34	122.702.34		5.1	3.179.499.30	3.179.499.30		36.2	195.8	834.989.24	78.6	471.208.12
Oregon	3.053.448	345.126.66	317.233.16		40.4	2.203.511.26	2.098.973.20	14.616.53	23.0	101.5	484.706.10	37.9	172.535.54
Pennsylvania	5.757.978	12.791.10	12.791.10		.2	3.985.025.14	3.985.025.14		11.6	97.2	1.446.164.14	26.6	313.997.62
Rhode Island	999.354	18.830.63	18.830.63		.5	969.153.21	969.153.21		25.9	20.0	10.468.37		901.79
South Carolina	2.729.583	1.740.69	1.740.69			1.886.219.50	1.883.489.88	2.729.62	26.6	184.4	282.842.64	22.5	561.509.79
South Dakota	3.005.739	871.013.41	865.096.92	5.916.49	145.0	1.101.467.48	1.069.443.19	32.024.29	6.3	177.2	467.166.15	50.2	604.032.74
Tennessee	4.246.309	131.887.64	72.591.00	59.296.64	5.1	2.571.906.04	2.055.185.30	516.720.74	22.9	98.5	1.257.905.72	48.7	860.626.98
Texas	12.122.012	375.515.25	301.232.95		97.5	7.151.369.87	6.072.823.35		23.9	616.3	2.451.598.23	269.9	3.296.357.47
Utah	2.374.205	869.670.21	869.670.21		101.5	1.032.219.87	1.031.824.97		25.3	80.9	300.059.76	12.1	172.650.06
Vermont	931.919	47.983.81	47.983.81		6.2	735.351.85	707.198.60		29.7	34.7			176.736.59
Virginia	3.708.379	150.795.89	133.613.02	17.182.87	10.2	1.586.019.40	1.585.607.09	412.31	31.5	56.1	1.565.216.55	60.1	423.942.34
Washington	3.057.934	202.645.88	202.645.88		13.7	2.105.073.42	2.101.716.60	3.356.82	18.3	75.4	370.607.20	14.9	382.964.32
West Virginia	2.013.405	144.034.00	144.034.00		4.9	1.522.383.88	1.522.383.88		40.5	64.5	261.526.91	2.8	85.460.21
Wisconsin	4.862.441	298.477.25	288.174.42	1.190.00	15.7	2.502.675.31	2.481.175.31	21.500.00	37.6	101.1	434.346.68	37.5	1.654.744 59
Wyoming	2.250.663	576.020.88	488.300.00	60.000.00	108.2	1.529.554.80	1.441.853.00	18.000.00	21.4	308.7	104.382.61	39.0	216.127.39
District of Columbia													
Hawaii	1.683.956	25.988.65	12.994.33	12.994.32	.9	1.328.218.44	1.096.391.33	181.827.36	4.4	27.9	281.437.50	4.0	893.132.84
TOTALS	185.768.083	11.440.029.22	10.597.631.87	457.404.58	1.700.3	117.322.766.49	104.499.947.61	6.710.872.14	26.9	6.319.6	34.587.028.36	2.358.3	36.083.475.16

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS II—PROJECTS ON EXTENSIONS OF THE FEDERAL-AID HIGHWAY SYSTEM INTO AND THROUGH MUNICIPALITIES

AS OF JANUARY 31,1934

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS II PROJECTS IN MUNICIPALITIES	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS II PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$			$	$	$			$		$
Alabama	2.092.533					283.370.58	283.370.58		31.8	7.7	145.359.80	5.6	1.663.802.62
Arizona	781.794	6.427.92	6.427.92		.6	48.580.21	43.999.66		8.1	1.6	107.565.46	1.7	623.800.96
Arkansas	1.687.084					449.712.15	381.118.60	68.593.55	20.7	11.5	233.226.16	7.6	1.072.739.24
California	3.901.839	61.839.78	55.291.30		2.2	1.797.975.84	1.578.030.15		11.2	22.9	961.170.55	12.8	1.307.347.00
Colorado	1.718.633	128.443.81	128.443.81		3.3	300.312.15	300.312.15		10.8	5.5	312.065.76	5.9	977.811.28
Connecticut	802.407	52.696.86	52.696.86		.5	430.230.83	430.230.83		30.3	6.9	255.987.28	2.3	63.492.03
Delaware	454.772	75.309.50	75.309.50		1.5	149.600.00	149.600.00		3.8	1.1	50.710.00	.8	179.152.50
Florida	1.307.959					637.053.01	429.287.80	207.765.21	26.8	7.2	879.671.20	10.3	
Georgia	2.724.620					363.826.39	363.826.39		18.7	10.3	306.740.42	16.5	2.054.053.19
Idaho	1.121.562	22.291.47	22.291.47		1.7	320.236.37	314.841.97		11.3	5.4	36.803.14	2.0	747.625.42
Illinois	6.877.199	45.749.72	45.749.72		.6	2.751.849.25	2.751.849.25		9.6	26.2	3.151.968.22	32.4	927.631.81
Indiana	4.818.165					24.698.54	24.698.54		4.0	.4	140.870.89	3.0	4.652.595.57
Iowa	2.815.585	253.708.74	242.350.00		8.6	657.911.44	618.150.00		17.1	14.6	705.915.00	17.2	1.249.170.00
Kansas	2.522.401	24.291.11	24.291.11		1.0	1.121.956.62	1.088.896.38		2.8	14.4	1.409.213.51	21.7	
Kentucky	2.029.687	13.307.85	13.307.85		.2	49.231.08	49.231.08		7.5	.7	352.411.28	9.9	1.614.736.79
Louisiana	1.457.148	32.826.75	32.826.75		.1	325.613.74	325.613.74		50.7	6.5	119.476.09	6.4	979.231.42
Maine	842.479	79.296.57	79.296.57		1.6	350.433.41	350.433.41		32.4	8.5	206.951.72	3.3	205.797.30
Maryland	891.132					16.788.76	12.638.76		79.1	.6			878.493.24
Massachusetts	5.007.199	53.896.49	30.896.49	23.000.00	1.5	3.009.497.66	2.995.397.66	14.100.00	13.3	12.2	310.455.80	1.1	1.670.449.05
Michigan	4.457.679	180.700.00	180.700.00		2.3	1.046.470.00	1.044.320.00		7.8	20.4	1.383.046.90	8.0	1.849.613.00
Minnesota	3.410.102	467.206.86	467.206.86		32.7	753.470.90	753.470.90		48.7	30.4	430.801.53	18.1	1.758.622.71
Mississippi	1.744.669					350.871.53	324.704.39	26.167.44	21.9	12.2	116.976.43	7.3	1.302.988.18
Missouri	3.045.077	4.587.13	3.305.88		.3	777.489.27	765.426.15		33.3	8.8	795.902.23	19.1	1.481.342.74
Montana	1.115.962	2.878.39	2.878.39		.1	359.459.22	359.459.22		19.0	9.6	104.507.43	8.1	649.116.96
Nebraska	1.957.240	29.267.12	28.577.83		3.6	896.362.68	896.362.68		28.3	16.2	343.325.63	8.3	688.973.86
Nevada	500.051	50.304.27	50.304.27		1.2								449.746.73
New Hampshire	706.640					318.013.03	318.013.03		12.6	6.9	346.352.92	8.7	42.274.05
New Jersey	3.217.442	70.309.86	70.309.86		.8	1.648.571.00	1.648.571.00		22.9	12.5	630.404.30	6.0	868.156.84
New Mexico	1.448.234	10.526.42	10.526.42		1.0	407.952.45	407.952.45		15.8	7.3	241.332.37	8.2	788.422.76
New York	7.337.365	129.750.00	129.750.00		2.2	4.513.330.00	4.392.045.00		16.4	36.5	2.759.588.00	20.8	556.482.00
North Carolina	2.380.573	48.650.33	48.650.33		3.7	135.891.79	135.891.79		32.6	8.5	264.663.05	12.1	1.931.367.53
North Dakota	1.451.112	2.207.35	2.207.35		1.1	49.955.24	49.955.24		33.6	6.2	303.890.76	11.5	1.095.058.15
Ohio	4.645.378	63.410.00	55.935.00		2.0	1.173.680.00	1.091.180.00		24.5	13.5	1.366.348.43	17.1	2.131.914.57
Oklahoma	2.304.200	18.612.59	18.612.59		.8	306.229.29	306.229.29		9.9	8.0	893.617.17	19.3	1.085.743.95
Oregon	1.526.724	23.731.40	23.731.40		.5	605.684.88	591.369.15	14.315.73	7.0	10.3	439.139.48	12.4	422.433.97
Pennsylvania	5.416.051	68.643.58	68.643.58		2.8	1.108.022.40	1.099.002.28		24.2	21.6	1.522.588.35	23.5	2.725.816.79
Rhode Island	499.677					155.048.79	155.048.79		2.6	2.4	106.304.22	1.8	238.323.99
South Carolina	1.364.791	24.919.48	24.919.48		1.7	211.930.64	211.525.63	405.01	10.0	14.4	224.461.30	7.5	905.884.59
South Dakota	1.502.370	75.816.40	75.816.40		5.2	158.605.22	158.605.22		12.7	5.9	190.209.51	7.2	1.078.238.87
Tennessee	2.123.155	19.992.06	19.992.06		1.1	245.739.64	245.739.64		19.9	4.3	504.342.86	8.3	1.353.080.44
Texas	6.061.006	57.311.09	48.312.46		5.4	1.459.189.34	1.313.309.79		16.9	54.0	1.510.295.51	31.3	3.188.597.94
Utah	771.326	351.502.28	351.425.28		8.8	187.168.55	186.309.55		28.2	4.9	62.132.42	2.0	172.052.75
Vermont	470.628	3.854.07	3.854.07		.6	298.777.26	297.549.64		21.6	8.2	128.245.15	2.9	40.979.14
Virginia	1.854.189	51.612.44	42.851.95	8.760.49	2.0	382.857.16	370.357.16		27.9	8.0	1.080.471.55	14.6	360.508.34
Washington	1.877.571	110.211.77	110.211.77		3.2	501.303.27	496.515.67		12.8	9.8	1.062.213.41	15.0	208.825.15
West Virginia	1.342.270	15.323.43	15.323.43		.4	136.737.74	136.737.74		18.6	4.0	426.277.66	7.8	713.931.17
Wisconsin	2.431.220	151.529.60	151.529.60		4.7	636.105.60	636.105.60		32.6	16.8	269.044.64	6.2	1.133.540.16
Wyoming	1.125.332	53.504.69	53.004.69		1.1	252.799.93	252.799.93		11.5	6.2	357.775.09	7.7	461.751.29
District of Columbia	959.235	295.584.85	295.584.85		2.4	570.729.00	570.729.00		6.0	1.7	92.921.15	.6	
Hawaii													
TOTALS	113.402.967	3.232.534.53	3.163.345.65	31.760.49	115.2	32.792.324.15	31.757.212.88	331.346.94	17.9	533.7	27.680.387.13	485.0	50.801.521.34

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION
AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

CLASS III—PROJECTS ON SECONDARY OR FEEDER ROADS

AS OF JANUARY 31,1934

STATE	PUBLIC WORKS FUNDS ASSIGNED FOR CLASS III PROJECTS ON SECONDARY HIGHWAYS	COMPLETED			UNDER CONSTRUCTION				APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR NEW CLASS III PROJECTS	STATE
		Total cost	Public works funds	Mileage	Estimated total cost	Public works funds allotted	Percentage completed	Mileage	Public works funds allotted	Mileage		
	$	$	$		$	$			$		$	
Alabama	2,092,533				53,918.15	53,918.15	3.7	.2	85,609.15	12.6	1,953,005.70	Alabama
Arizona	625,435				379,056.78	315,700.00	8.9	31.8	54,830.68	.3	254,904.32	Arizona
Arkansas	1,687,084				293,499.27	293,499.27	3.1	15.0	194,488.38	34.2	1,199,096.35	Arkansas
California	3,901,838				1,581,781.61	1,245,217.35	14.0	109.0	1,020,964.69	32.3	1,635,655.96	California
Colorado	1,718,632	50,000.00	50,000.00	12.2	639,262.42	639,262.42	62.4	111.4	300,354.84	32.2	729,014.74	Colorado
Connecticut	659,120				664,915.58	659,120.00	2.0	14.5				Connecticut
Delaware	454,772								138,752.00	1.8	316,020.00	Delaware
Florida	1,307,958				1,097,261.16	1,097,261.16	16.7	65.0	120,441.97	3.4	90,254.87	Florida
Georgia	2,320,973				102,200.54	102,200.54	11.7	12.5	392,307.88	15.2	1,826,464.58	Georgia
Idaho	1,121,562	54,025.34	49,923.08	6.8	824,240.13	792,218.80	37.6	117.8			279,420.12	Idaho
Illinois	6,262,223	22,161.15	22,161.15	3.5	1,235,769.20	1,235,769.20	12.0	126.7	1,895,287.26	117.8	3,109,005.39	Illinois
Indiana	501,892				165,000.00	165,000.00	10.3	48.1	151,464.91	11.6	185,427.09	Indiana
Iowa	2,212,245	59,388.56	58,100.00	10.0	494,851.23	470,900.00	42.5	45.3	354,000.00	54.7	1,389,245.00	Iowa
Kansas	2,522,401				658,119.38	658,119.38	3.8	71.4	1,864,281.62	113.5		Kansas
Kentucky	1,879,340				1,169,215.17	1,169,215.17	10.4	154.0	664,724.04	69.0	45,400.79	Kentucky
Louisiana	1,457,148				371,567.82	371,567.82	15.3	15.5	263,439.46	17.5	822,140.72	Louisiana
Maine	842,479	564,255.93	560,104.50	68.0	318,148.97	265,305.68	77.2	29.8	8,989.70	.2	8,079.12	Maine
Maryland	891,132				179,884.07	179,884.07	12.2	12.9	271,118.20	21.7	440,129.73	Maryland
Massachusetts	488,185				469,741.41	469,741.41	34.9	15.2			18,443.59	Massachusetts
Michigan	3,184,057				1,831,000.00	1,831,000.00	5.6	142.2	958,750.00	90.5	394,307.00	Michigan
Minnesota	2,131,314	145,774.08	145,774.08	56.9	1,186,066.92	1,105,635.92	21.1	161.9	264,600.02	50.1	615,103.98	Minnesota
Mississippi	1,744,669				435,000.00	435,000.00	34.7	54.6			1,309,669.00	Mississippi
Missouri	3,045,076				1,998,477.63	1,998,477.63	19.5	364.1	812,022.93	92.7	234,575.44	Missouri
Montana	1,859,937	21,033.70	21,033.70	2.3	663,065.04	663,065.04	4.8	71.3	449,247.61	59.1	726,590.65	Montana
Nebraska	1,957,240				1,757,064.92	1,755,202.60	21.6	250.3	81,969.21	7.0	120,068.19	Nebraska
Nevada	1,136,479				852,640.45	852,640.45	23.1	97.0			283,838.55	Nevada
New Hampshire	477,460				494,972.03	453,460.00	23.8	25.1	24,000.00	.5		New Hampshire
New Jersey	63,460				56,550.52	56,550.52	3.5	.5			6,909.48	New Jersey
New Mexico	1,448,234	9,000.00	9,000.00	4.0	1,071,950.00	1,071,950.00	31.3	697.7	63,900.00	73.0	303,384.00	New Mexico
New York	3,662,137	144,300.00	144,300.00	4.6	3,809,913.19	3,378,488.19	14.8	93.1	94,475.00	1.9	44,873.81	New York
North Carolina	2,380,573	136,533.26	136,533.26	16.3	1,023,427.93	1,023,427.93	25.1	89.2	74,924.30	11.3	1,145,687.51	North Carolina
North Dakota	1,451,112								46,035.01	10.4	1,405,076.99	North Dakota
Ohio	3,871,148	71,000.00	71,000.00	30.5	2,424,420.00	2,318,984.90	22.4	242.7	907,210.00	29.1	573,953.10	Ohio
Oklahoma	2,304,199								647,185.02	93.4	1,657,013.98	Oklahoma
Oregon	1,526,724				592,391.68	591,391.68	7.1	20.2	843,733.39	69.1	91,598.93	Oregon
Pennsylvania	7,716,975	32,266.00	32,266.00	4.0	5,949,384.85	5,894,946.40	23.4	581.7	1,056,704.43	89.1	733,058.17	Pennsylvania
Rhode Island	499,677				70,179.89	70,179.89	1.4	6.0	253,908.32	20.1	175,588.79	Rhode Island
South Carolina	1,364,791				1,157,796.30	1,157,796.30	16.0	135.8	65,649.83	3.0	141,344.87	South Carolina
South Dakota	1,502,870	39,421.46	39,421.46	16.1	43,072.35	43,072.35	68.7	14.8	119,208.50	45.9	1,301,167.69	South Dakota
Tennessee	2,123,155				369,461.70	369,461.70	22.5	38.8	864,749.54	67.6	888,943.76	Tennessee
Texas	6,061,006	13,451.11	6,850.00	8.9	4,121,840.85	3,209,592.90	27.2	540.8	1,387,049.54	192.5	1,457,513.56	Texas
Utah	1,048,677	61,543.36	61,543.36	8.0	758,171.26	757,124.24	39.0	136.9	106,733.22	6.0	123,276.18	Utah
Vermont	465,026				327,773.35	309,469.49	18.7	28.3	28,124.48	1.9	127,432.03	Vermont
Virginia	1,854,189	16,000.00	16,000.00	11.7	1,301,621.03	1,260,738.44	26.2	186.8	249,388.95	10.8	328,061.61	Virginia
Washington	1,180,362	36,856.60	36,856.60	11.6	873,623.16	863,623.16	12.5	43.4	115,050.50	10.1	164,831.74	Washington
West Virginia	1,118,559				397,153.15	397,153.15	10.3	23.0	399,486.33	22.2	321,919.52	West Virginia
Wisconsin	2,431,220	26,056.01	26,056.01	.6	1,449,830.75	1,410,815.47	20.6	94.8	313,193.38	25.6	681,155.14	Wisconsin
Wyoming	1,125,332	32,972.04	31,300.00	4.1	576,809.16	565,815.00	37.6	88.7	154,396.65	20.3	373,820.35	Wyoming
District of Columbia	959,234	116,450.00	116,450.00	2.0	681,145.93	658,254.90	14.2	4.4	184,529.10	1.4		District of Columbia
Hawaii	187,106								180,042.06	4.9	7,063.94	Hawaii
TOTALS	94,828,950	1,652,488.60	1,634,673.20	282.1	46,973,236.93	44,687,218.67	20.8	5,230.7	18,527,522.10	1,647.5	29,979,536.03	TOTALS

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D.C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1927. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1928. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1929. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1932. 10 cents.

DEPARTMENT BULLETINS

No. 136D . . Highway Bonds. 20 cents.

No. 347D . . Methods for the Determination of the Physical Properties of Road-Building Rock. 10 cents.

No. 532D . . The Expansion and Contraction of Concrete and Concrete Roads. 10 cents.

No. 583D . . Reports on Experimental Convict Road Camp, Fulton County, Ga. 25 cents.

No. 660D . . Highway Cost Keeping. 10 cents.

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

TECHNICAL BULLETINS

No. 55T . . Highway Bridge Surveys. 20 cents.

No. 265T . . Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS CIRCULARS

No. 62MC . . Standards Governing Plans, Specifications Contract Forms, and Estimates for Federal Aid Highway Projects. 5 cents.

No. 93MC . . Direct Production Costs of Broken Stone. 25 cents.

MISCELLANEOUS PUBLICATION

No. 76MP . . The results of Physical Tests of Road-Building Rock. 25 cents.

No. . . Federal Legislation and Regulations Relating to Highway Construction. 10 cents.

REPRINT FROM PUBLIC ROADS

Reports on Subgrade Soil Studies. 40 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U.S. Bureau of Public Roads, Willard Building, Washington, D.C.

CURRENT STATUS OF U.S. PUBLIC WORKS ROAD CONSTRUCTION

AS PROVIDED IN TITLE II, SECTION 204 OF THE NATIONAL INDUSTRIAL RECOVERY ACT

SUMMARY OF CLASSES I, II, AND III

AS OF JANUARY 31,1934

STATE	TOTAL APPORTIONMENT OF PUBLIC WORKS FUNDS	COMPLETED				UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION		BALANCE OF PUBLIC WORKS FUNDS AVAILABLE FOR ALL NEW PROJECTS
		Total cost	Public works funds	Regular Federal aid	Mileage	Estimated total cost	Public works funds allotted	Regular Federal aid allotted	Percentage completed	Mileage	Public works funds allotted	Mileage	
	$	$	$	$		$	$	$			$		$
Alabama	8.370.133					4.779.023.49	2.788.396.85	1.990.626.64	26.4	258.8	823.967.57	76.4	4.757.768.58
Arizona	5.211.960	311.966.53	311.966.53		22.2	3.476.998.86	3.026.779.65		24.8	261.4	344.718.27	5.3	1.528.495.55
Arkansas	6.748.335	18.339.52	9.849.52	8.490.00	.6	2.142.209.67	1.694.653.75	447.555.92	26.4	83.4	1.020.648.01	84.4	4.023.183.72
California	15.607.354	153.985.20	129.694.50		4.0	10.942.699.40	8.717.322.47		20.4	359.6	3.114.134.75	89.0	3.646.202.28
Colorado	6.874.530	679.330.81	679.330.81		48.8	2.422.268.08	2.411.687.73		39.0	185.4	908.215.88	50.1	2.875.295.58
Connecticut	2.865.740	52.696.86	52.696.86		.5	2.326.048.13	2.144.473.61	175.778.94	9.4	45.8	565.835.77	8.6	102.733.76
Delaware	1.519.088	208.841.80	208.841.80		16.9	892.415.70	892.415.70		18.4	18.5	189.462.00	2.6	528.368.50
Florida	5.231.834					4.306.979.87	3.315.330.45	988.039.32	27.1	164.8	1.554.760.60	32.0	358.742.95
Georgia	10.091.185					2.225.472.25	2.225.472.25		30.2	118.9	1.822.498.35	109.2	6.043.214.40
Idaho	4.486.249	350.816.30	345.485.12		44.0	2.503.380.61	2.452.460.73		37.5	218.3	183.322.04	15.4	1.504.981.11
Illinois	17.570.770	67.910.87	67.910.87		4.1	4.910.434.48	4.910.434.48		10.3	174.7	5.611.110.53	158.4	6.981.314.12
Indiana	10.037.843					2.164.115.34	2.164.115.34		27.0	116.0	1.416.538.44	46.8	6.457.189.22
Iowa	10.055.660	1.020.135.11	1.004.550.00		48.6	5.100.857.10	4.808.750.00		29.7	269.7	1.552.395.00	94.7	2.689.665.00
Kansas	10.089.604	173.948.41	173.948.41		50.5	4.879.916.41	4.842.895.04		22.0	371.2	5.072.760.55	201.6	
Kentucky	7.517.359	103.886.95	103.886.95		3.1	3.041.677.48	3.041.677.48		23.9	296.8	2.172.645.14	166.8	2.199.149.43
Louisiana	5.828.591	45.452.82	45.452.82		.5	2.723.526.60	2.216.875.60		25.2	59.0	1.167.467.36	54.4	2.398.795.22
Maine	3.369.917	643.552.50	639.401.07		69.6	1.829.903.68	1.741.493.44		41.9	73.8	434.011.58	10.9	555.010.91
Maryland	3.564.527					997.816.61	984.017.86		8.9	28.3	271.118.20	21.7	2.309.390.94
Massachusetts	6.597.100	207.833.37	114.849.13	92.984.24	5.7	4.591.601.53	4.261.841.89	329.759.64	16.5	60.2	423.153.99	1.9	1.797.254.99
Michigan	12.736.227	216.200.00	216.200.00		3.5	5.327.020.00	5.324.870.00		18.9	893.2	4.618.662.00	186.5	2.576.495.00
Minnesota	10.656.569	1.845.260.53	1.845.260.53		346.8	3.758.847.63	3.678.416.61		31.2	458.6	1.210.860.27	209.4	3.922.031.59
Mississippi	6.978.675					3.695.807.94	2.324.200.01	1.332.975.99	16.8	211.8	577.751.84	64.0	4.076.723.15
Missouri	12.180.306	322.760.82	222.812.20		20.4	6.095.614.48	5.768.525.52	25.499.64	33.0	478.1	3.154.007.34	168.1	3.034.960.94
Montana	7.439.748	241.914.17	241.914.17		36.4	4.731.329.66	4.458.450.78	192.878.88	26.8	349.4	1.210.358.50	107.9	1.529.024.55
Nebraska	7.828.961	410.391.09	283.998.37		148.4	6.453.070.85	5.596.975.42	108.538.00	31.9	442.8	982.982.95	72.6	965.004.26
Nevada	4.545.917	322.539.87	322.539.87		61.5	2.319.345.93	2.319.345.93		22.7	228.4			1.904.031.20
New Hampshire	1.909.839					1.358.320.37	1.311.123.34		23.0	41.2	469.473.01	10.5	129.242.65
New Jersey	6.346.039	70.309.86	70.309.86		.8	2.803.677.08	2.803.677.08		17.2	32.3	1.476.322.29	16.3	1.995.729.77
New Mexico	5.792.935	1.092.305.73	1.092.305.73		144.0	2.790.965.68	2.790.965.68		15.5	799.3	731.496.48	125.6	1.178.167.11
New York	22.330.101	391.150.00	391.150.00		10.6	17.526.093.19	15.995.708.19	315.000.00	20.1	331.4	4.712.293.00	52.3	1.230.949.81
North Carolina	9.522.293	629.883.69	407.533.67	222.350.02	55.1	2.448.658.52	2.119.000.82	329.657.70	29.5	246.8	1.918.075.06	242.1	5.077.683.45
North Dakota	5.804.448	502.968.94	502.965.43		250.8	911.033.06	911.033.06		12.2	236.8	1.515.220.58	377.5	2.875.228.93
Ohio	15.484.592	347.780.00	340.305.00		37.3	9.284.365.00	8.978.479.90		30.1	403.7	3.157.533.43	67.6	3.008.273.67
Oklahoma	9.216.798	141.314.93	141.314.93		5.9	3.485.728.59	3.485.728.59		33.9	203.8	2.375.791.43	191.3	3.213.963.05
Oregon	6.106.896	368.858.06	340.964.56		41.0	3.401.587.82	3.281.734.03	28.932.26	17.2	132.0	1.797.628.97	119.4	686.568.44
Pennsylvania	18.891.004	113.700.68	113.700.68		7.0	11.042.432.39	10.978.973.82		19.2	700.5	4.025.456.92	139.3	3.772.872.58
Rhode Island	1.998.708	18.830.63	18.830.63		.5	1.194.381.89	1.194.381.89		21.4	28.4	370.680.91	21.9	414.314.57
South Carolina	5.459.165	26.660.17	26.660.17		1.7	3.255.946.44	3.252.811.81	3.134.63	21.7	334.6	570.953.77	33.0	1.608.739.25
South Dakota	6.011.479	986.251.27	980.334.78	5.916.49	166.3	1.303.145.05	1.271.120.76	32.024.29	9.2	197.9	776.584.16	103.3	2.983.439.30
Tennessee	8.492.819	151.879.70	92.583.06	59.296.64	6.2	3.187.107.38	2.670.386.64	516.720.74	22.6	141.6	2.626.998.12	124.6	3.102.651.18
Texas	24.244.024	446.777.45	356.395.41		111.8	12.732.400.06	10.596.226.04		24.0	1.211.1	5.348.933.58	493.7	7.942.468.97
Utah	4.194.708	1.282.715.85	1.282.638.85		118.3	1.977.559.68	1.975.158.76		30.9	222.7	468.925.40	20.1	467.984.99
Vermont	1.867.573	51.837.88	51.837.88		6.8	1.361.902.46	1.314.217.73		25.3	71.7	156.369.63	4.8	345.147.76
Virginia	7.416.757	218.408.33	192.464.97	25.943.36	23.9	3.270.497.59	3.216.702.69	412.31	29.0	250.9	2.895.077.05	85.5	1.112.512.29
Washington	6.115.867	349.714.25	349.714.25		28.5	3.479.999.85	3.461.855.43	3.356.82	16.1	128.6	1.547.876.11	43.0	756.421.21
West Virginia	4.474.234	159.357.43	159.357.43		5.3	2.106.274.77	2.106.274.77		32.9	91.5	1.087.290.90	32.8	1.121.310.90
Wisconsin	9.724.881	476.062.86	465.760.03	1.190.00	21.0	4.588.511.66	4.528.096.38	21.500.00	31.6	212.7	1.007.584.70	69.3	3.723.439.89
Wyoming	4.501.327	682.497.61	572.604.69	60.000.00	113.4	2.359.163.89	2.260.467.93	15.000.00	24.3	403.6	616.555.35	67.0	1.051.699.03
District of Columbia	1.918.469	412.034.85	412.034.85		4.4	1.251.874.93	1.228.983.90		10.4	6.1	277.450.25	2.0	
Hawaii	1.871.062	25.988.65	12.994.33	12.994.32	.9	1.328.218.44	1.096.391.33	151.827.36	4.4	27.9	461.479.56	8.9	300.196.78
TOTALS	394.000.000	16.325.052.35	15.395.650.72	489.165.07	2.097.6	197.083.327.57	180.944.379.16	7.042.219.08	23.8	12.084.0	80.795.437.59	4.490.8	116.864.532.53

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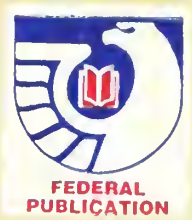

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